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8.1 INTRODUCTION

The methods of hydrologic and hydraulic analysis provided in this chapter give the designer information necessary for an analysis of a roadway drainage crossing. Experience and sound engineering judgment are not to be ignored and may, at times, differ from results obtained using methods in this chapter. Very careful weighing of experience, judgment, and procedure must be made to arrive at a solution to the problem. Research in the field of drainage continues throughout the country and may subsequently alter the procedures found in this chapter.

(1) Objectives of Highway Drainage

The objective of highway drainage is to prevent the accumulation and retention of water on and/or around the highway by:

a. Anticipating the amount and frequency of storm runoff.

b. Determining natural points of concentration of discharge and other hydraulic controls.

c. Removing detrimental amounts of surface and subsurface water, and

d. Providing the most efficient hydraulic design consistent with economy, the importance of the road, maintenance and legal obligations.

(2) Basic Policy

In designing highway drainage, there are three major considerations; first, the safety of the traveling public, second, the design should be in accordance with sound engineering practices to economically protect and drain the highway, and third, in accordance with reasonable interpretation of the law, to protect private property from flooding, water soaking or other damage. In general, the hydraulic adequacy of structures is determined by the methods as outlined in this manual and performance records of structures in the same or similar locations.

(3) Design Frequency

Federal and State governments have placed increasing emphasis on environmental protection over the last several years. Consequently the administrative rules established by regulatory agencies have made past practice of designing structures to accommodate flood frequencies of 25 and 50 years obsolete and unworkable. Thus, the design discharge for all bridges and box culverts covered under this chapter shall be the 100 year (Q_{100}) frequency flood. In floodplain management this is also referred to as the Regional or Base flood. Design frequency is determined from requirements in Federal Highway Administration (FHWA) directives and the co-operative agreement between Wisconsin Department of Transportation (DOT) and Wisconsin Department of Natural Resources (DNR). The following publications are suggested for guidance.

a. FHWA Directive

Volume 6, Chapter 7, Section 3, Subsection 2, of the FHWA - Federal Aid Highway Program Manual, "Location and Hydraulic Design of Encroachments on Flood Plains", prescribes FHWA policy and procedures. Copies of this directive may be obtained from any of the Division of Transportation System Development Regional offices.

b. DNR-DOT Cooperative Agreement

The Wisconsin Department of Transportation and the Wisconsin Department of Natural Resources have signed a co-operative agreement to provide a reasonable and economical procedure for carrying out their respective duties in a manner that is in the total public interest. See Appendix 8-B. The provisions in this agreement establish the basic considerations for highway stream crossings.

c. DOT Facilities Development Manual

Refer to Procedures - Chapter 13 - Drainage Practice,
Chapter 20 - Environmental Laws, Policies and Regulations and Chapter 21 - Environmental Documents, Reports and Permits.

(4) Hydraulic Site Report

A hydraulic report for all projects shall be submitted with the "Stream Crossings Structure Survey Report" for Bridges and Box Culverts. A sample hydraulic report is included in Appendix 8-A. Plan survey datum must conform to datum in use by local zoning authorities. In most cases elevations are referenced from the National Geodetic Vertical Datum (NGVD) of 1929. This report discusses and documents the hydrologic, hydraulic, site conditions, and all other pertinent factors that influence the type, size, and location of the proposed structure.

(5) Hydraulic Design Criteria for Temporary Structures

The basic design criteria for temporary structures will to be the ability to pass a 5-year storm (Q5) with only 0.5 feet of backwater over existing conditions. This criteria is only a general guideline and site specific factor and engineering judgment may indicate that this criteria is inappropriate. Factors that should be considered in the design of temporary structures and embankments are:

1. Evaluate effect on surrounding property and buildings.
2. Velocities that would cause excessive scour.
3. Damage or inconvenience due to failure of temporary structure.
4. DNR concerns.
5. Temporary roadway profile.
6. Structure depths will be 36” for short spans and 48” or more for longer spans.

If possible and practical, the temporary roadway profile should be designed and constructed in such a manner that infrequent flood events are not obstructed from overflowing the temporary profile and creating excessive backwaters upstream of the construction. The temporary roadway profile should provide adequate clearance for the temporary structure.

The roadway designer should indicate the need for a temporary structure on the Stream Crossing Structure Survey Report.

Preliminary and Final plans should indicate the hydraulic parameters of the temporary structure. The required parameters are the 5-year flood discharge (Q5), the 5-year high-water elevation (HW5), and the flow area of the temporary structure required to pass the 5-year flood (Abr).

(6) Erosion Control Parameters

In order to assist designers in determining the appropriate erosion control measures to be provided at Bridge construction site, preliminary and final plans should indicate the 2-year flood discharge (Q2) and the 2-year high-water elevation (HW2).

(7) Bridge Rehabilitation and Hydraulic Studies

Generally no hydraulic study will be required in bridge rehabilitation projects that do not involve encroachment to the Base Floodplain. This includes entire super structure replacement provided that the substructure and berm configuration remain unchanged and the low cord elevation is not significantly lowered.

The designer should consider historical high-water elevations, Flood Insurance Studies and the potential of inundation when choosing the replacement superstructure type. The risk of damage to the structure as the result of Scour should also be considered.

8.2 HYDROLOGIC ANALYSIS

The first step in designing a hydraulic structure is to determine the design discharge for the waterway. The problem is particularly difficult for small watersheds, say under five square miles, because the smaller the area, the more sensitive it is to conditions which affect runoff and the less likely there are runoff records for the area.

Acceptable methods of determining the design discharge for the 100 year flood shall be based on the guidelines contained in S.NR 116.07, *Wisconsin's Floodplain Management Program* (1). Generally, **a minimum of two methods** should be used in determining a design discharge.

Hydrology and the Metric Units

Most of the frequently used methodologies for hydrologic calculations are based on English units and input parameters. Therefore, for Metric design applications, most hydrologic calculations will be conducted in English units and "soft" converted to Metric values for use in hydraulic applications.

The most frequently used methods for determining the design discharge for bridges and box culverts in the State of Wisconsin are discussed below.

(1) Regional Regression Equations

The U. S. Geological Survey (USGS) in cooperation with the Wisconsin Department of Transportation prepared a report entitled **Flood Frequency Characteristics of Wisconsin Streams (2)** which considers the flood potentials for a site using regional regression equations based on flood data from gaging stations on Wisconsin's rivers and streams. The flood-frequency regression equations are correlated with three or more of seven parameters, namely, drainage area, main-channel slope, storage, forest cover, mean annual snowfall, precipitation intensity index, and soil permeability. These equations are applicable to all drainage areas in Wisconsin except for highly regulated streams, main stems of rivers given separate treatment, and highly urbanized areas of the state.

(2) Watershed Comparison

The results obtained from the above regression equations should be compared to similar gaged watersheds listed in reference (2) above using the area transfer formulas and procedures detailed in that document. A good discussion and examples of the use of regression equations and basin comparison methods can be seen in the WisDOT Facilities Development Manual, Procedure 13-10-5.

The flood frequency discharges listed in reference (2) are for flood records up to 2000. More years of data are available from the USGS for most of the gaged watersheds.

The flood frequency discharges for the gaged watersheds can be updated past water year 2000 by using the Log-Pearson Type III distribution method as described in Bulletin #17B entitled Guidelines For Determining Flood Flow Frequency (3)(4) and the guidelines for weighting the station skew with the generalized skew in S.NR116.07, Wisconsin's Floodplain Management Program (1).

(3) Flood Insurance Studies

The Federal Emergency Management Agency (FEMA) had contracted for detailed flood studies throughout Wisconsin. They were developed for floodplain management and flood insurance purposes. These Flood Insurance Studies (FIS) on file with Floodplain-Shoreland Management Section of the Wisconsin Dept. of Natural Resources (DNR) contain discharge values for many sites. These studies, along with other various floodplain studies, may be obtained from the DNR's Floodplain Analysis Interactive Map by using the following link: http://www.dnr.state.wi.us/org/water/wm/dsfm/section/mapindex.htm

(4) Soil Conservation Service

For small watersheds in urban and rural areas, the Soil Conservation Service (SCS) has developed procedures to calculate storm runoff volumes, peak rates of discharge, hydrographs and storage volumes. The procedure is documented in *Technical Release 55 Urban Hydrology for Small Watersheds* (5).

(5) References

1) Wisconsin Department of Natural Resources, *Wisconsin's Floodplain Management Program*, Chapter NR116, Register, August 2004, No. 584.

2) U. S. Geological Survey, *Flood-Frequency Characteristics of Wisconsin Streams*. Water-Resources Investigations Report 03-4250, 2003. (Report can be found on the USGS web site using the following link: http://wi.water.usgs.gov/projects/flood/index.html)

3) United States Water Resources Council, *Guidelines for Determining Flood Flow Frequency, Bulletin #17B*, Revised September 1981.

4) U. S. Geological Survey, *Guidelines for Determining Flood Flow Frequency, Bulletin #17B Revised September 1981, Editorial Corrections*, March 1982.

5) U.S. Department of Agriculture, Soil Conservation Service, *Urban Hydrology for Small Watersheds, Technical Release 55* (2nd Edition), June 1986.

8.3 HYDRAULIC DESIGN OF BRIDGES

Bridge design for roadway stream crossings requires analysis of the hydraulic characteristics for both the "existing conditions" and the "proposed conditions" of the project site. A thorough hydraulic analysis is essential to providing a properly sized, safe and economical bridge design and assessing the relative impact that the proposed bridge has on the floodplain. The following subsections discuss design considerations and hydraulic design procedures for bridges. See Appendix 8-A for a sample Site - Hydraulic report which discusses the hydraulic design aspects of the bridge as well as other site characteristics that influenced the selection of the proposed structure.

(1) Hydraulic Design Factors

Several hydraulic factors dictate the design of both the bridge and the approach roadway within the floodplain limits of the project site. The critical hydraulic factors for design consideration are:

A. Velocity

Velocity through the bridge opening is a major design factor. Velocity relates to the scour potential in the bridge opening and the development of scour areas adjacent to the bridge. Examination of the "existing conditions" model, existing site conditions, soil conditions, and flooding history will give good insight to acceptable design velocity. Generally, velocities through bridges of less than 10 feet per second are acceptable. However, velocities up to 14 feet per second may be adequately addressed with heavy riprap protection.

B. Roadway Overflow

The vertical alignment of the approach grade is a critical factor in the bridge design when roadway overflow is a design consideration. The two important design features of roadway overflow is overtopping velocity and overtopping frequency. See Section 8.3(2)(F-2).

C. Bridge Skew

When a roadway is at a skew angle to the stream or floodway, the bridge shall also be at a skew to the roadway with the abutments and piers parallel to the flow of the stream. The hydraulic section through the bridge shall be the skewed section normal to the flow of the stream. Generally, in the design of stream crossing, the skew of the structure should be varied in increments of 5 degrees where practical. Improper skew can greatly aggravate the magnitude of scour.

D. Backwater and High-water Elevation

Roadways and bridges are generally restrictions to the normal flow of floodwaters and increase the flood profile in most situations. The increase in the flood profile is referred to as the backwater and the resultant upstream water surface elevation is referred to as the High-water Elevation (HW).

The high-water elevation or backwater calculations at the bridge are directly related to the bridge size and roadway alignment, which dictates all of the aforementioned hydraulic design factors. A significant design consideration when computing backwater is the potential for increasing flood damage for upstream property owners. The Cooperative Agreement between the Wis. Department of Natural Resources (DNR) and Wis. Department of Transportation (DOT) (Appendix 8-B) defines the policy for high-water elevation design. That portion of the Cooperative Agreement relating to floodplain considerations is based on the Wisconsin Adm. Rule NR116, "Wisconsin Floodplain Management Program". It is advisable to thoroughly study both documents as they can significantly influence the hydraulic design of the bridge.

One very subtle backwater criteria which is not addressed under the guidelines of the DNR-DOT Cooperative Agreement, is the backwater produced for flood events less than the 100 year frequency flood. Design consideration should be given to the more frequent flood events when there is potential for increasing the extent and frequency of flood damage upstream.

E. Freeboard

Freeboard is defined as the vertical distance between the low cord elevation of the bridge superstructure and the high-water elevation. A freeboard of 2.0 feet is the desirable minimum for all types of superstructures. However, economics, vertical and horizontal alignment, and the scope of the project may force a compromise to the 2 foot minimum freeboard. For these situations, close evaluation shall be made of the type and amount of debris and ice that would pass through the structure.

It has become common practice that if debris and ice are a potential problem, or adequate freeboard can not be provided, a concrete slab superstructure is preferred. A girder superstructure may be susceptible to damage when ice and/or debris is a significant problem. Girder structures are more susceptible to damage associated with buoyancy and lateral hydrostatic forces. Recent updates to policy have provided for the use of Precast Pretensioned Slab and Box

Sections where desirable freeboard can not be provided and conventional cast in place slabs can not be employed. Updates to the policy include the following:

1. Precast Pretensioned Slab and Box Sections may be in the water for the 100-year flood. The designer will be responsible for ensuring the stability of the structure for buoyant and lateral forces.

2. If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 5-year event, the Precast Pretensioned Slab and Box Sections must be cast solid.

3. If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 100-year event, the void in Precast Pretensioned Slab and Box Sections must be cast with a non water absorbing material.

F. Scour

Investigation of the potential for scour at the bridge site is a design consideration for the bridge opening geometry and size, as well as pier and abutment design. Bridges shall be designed to withstand the effects of scour from a super-flood (a flood exceeding the 100-year flood) without failing; i.e., experiencing foundation movement of a magnitude that requires corrective action. See Section 8.3(2)(G). Generally, scour associated with a 100-year event without significant reduction in foundation factor of safety will accomplish this objective. For situations where a combination of flow through a bridge and over the roadway exist, scour should also be evaluated for flow conditions at the onset of flow over topping when velocity through the bridge may be the greatest.

(2) Design Procedures

A. Determine Design Discharge

See Section 8.2 for procedures.

B. Determine Hydraulic Stream Slope

The primary method of determining the hydraulic slope of a stream is surveying the water surface elevation through a reach of stream 1500 feet upstream to 1500 feet downstream of the site. Intermediate points through this reach should also be surveyed to detect any significant slope variation.

There are situations, particularly on flat stream profiles, where it is difficult to determine a realistic slope using survey data. This will occur at normal water surface elevation at the mouth of a stream, upstream of a dam, or other significant restriction in the stream. In this case a USGS 7-1/2” quadrangle map and existing flood studies of the stream can be investigated to determine a reasonable stream slope.

C. Select Floodplain Cross-Section(s)

Generally, a minimum of two floodplain valley cross-section are required to perform the hydraulic analysis of a bridge. The section shall be normal to the stream flow at flood stage and approximately one bridge length upstream and downstream of the structure. A detailed cross-section of one or both faces of the bridge will also be required. If the section is skewed to the flow, the horizontal stationing shall be adjusted using the cosine of the skew angle.

Field survey cross-sections will be needed when a contour map is plotted using stereographic methods. A field survey section is needed for that portion below the normal water surface.

Cross-sections taken from contour maps are acceptable when the information is supplemented with field survey sections and data. Additional sections may be required to develop a proper hydraulic model for the site.

The hydraulic cross-sections should not include slack water portions of the flood plain or portions not contributing to the downstream movement of water.

Plot the elevation view of the Section(s) on cross-section paper for ease in dividing into sub-areas and developing a pictorial view of the section(s). Refer to *Facilities Development Manual* Procedure 9-55-5 for a discussion of Drainage Structure Surveys.

D. Assign “Manning n” Values to Section(s)

“Manning n” values are assigned to the cross-section sub-areas. Generally, the main channel will have different “manning n” values than the overbank areas. Values are chosen by on-site inspection, pictures taken at the section, and use of aerial photos defining the extent of each “n” value. There are several published sources on open channel hydraulics which contain tables for selecting appropriate “n” values. See references (1) and (2).

E. Select Hydraulic Model Methodology

There are several public and private computer software programs available for modeling open channel hydraulics, bridge hydraulics, and culvert hydraulics. Three public domain computer software programs that are most prevalent and preferred in Wisconsin bridge design work are “WSPRO”, “HY8”, and “HEC-2 (HEC-RAS)”.

The WSPRO methodology is tailored specifically for bridge hydraulics with many appropriate default coefficients and analysis options. The HEC-2 program and its successor HEC-RAS should be used where existing HEC-2 data is available from a previous Flood Insurance Study or when there is a need for a more sophisticated floodplain model. HEC-2 & HEC-RAS have more options and capabilities when modeling complex floodplains than WSPRO and require a greater amount of expertise to correctly apply. More information of these two programs is given below. “HY-8” is a FHWA sponsored culvert analysis package based the FHWA Publication “Hydraulic Design of Highway Culverts” (HDS-5).

WSPRO

“Water Surface Profiles (WSPRO)” is a computer software program developed by the U.S. Geological Survey under contract with the Federal Highway Administration. WSPRO was specifically oriented toward hydraulic design of highway bridges although it is equally suitable for water surface profile computations unrelated to highway and bridge design. The program uses conventional techniques for computing a gradually-varied flood profile as in other step-backwater models. However, bridge backwater computations are based on relatively recent developments in backwater analysis by USGS in a publication entitled Measurement of Peak Discharge at Width Contractions by Indirect Methods (3). Recent (1998) updates to the WSPRO program include full Metric capabilities, fully automated Scour analysis, and Floodway analysis.

A PC version of the WSPRO program and supporting documents are available through the “Mctrans Center” at the University of Florida at Gainesville. (See Appendix 8-C). For a complete treatise on the methodology of the program refer to reference (4).

HY8

HY8 is a program that presents FHWA publication HDS-5 procedures for analysis and design of highway culverts, design of energy dissipaters, storm hydrograph generation, and reservoir routing upstream of a culvert. Culvert hydraulics computations for circular, rectangular, elliptical, metal box, high and low profile arch, and arch shapes as well as for a user-defined geometry are performed. This methodology is discussed in Section 8.4.2(D). - Hydraulic Design of Box Culverts.

HEC-2 & HEC-RAS

The U.S. Army Corps of Engineers developed the water surface profile software program “HEC-2” in the early 1970’s. It was a very popular program with the private sector as well as local agencies involved in water resource engineering. The primary use was to develop detailed Flood Insurance Studies (FIS) for the National Flood Insurance program. It has not been widely used in Wisconsin for hydraulic bridge design. However, it has been used for bridge design where the hydraulic model for a detailed flood insurance study is available. HEC-RAS is the descendant of HEC-2. HEC-RAS is part of the Corps of Engineers “Next Generation” software application.

For a complete treatise on the methodology of the program, see reference (5) & (5a). The PC version of the program and supporting documents are also available through the McTrans Center.

F. Develop Hydraulic Model

First, a hydraulic model shall be developed for the “existing conditions” at the bridge site. This shall become the basis for hydraulic design of “proposed conditions” for the project and allows for an assessment of the relative hydraulic changes associated with the proposed structure. Special attention should be given to historic high-water and flood history, evidence of scour (high velocity), roadway overtopping, existing high-water, and compatibility with existing Flood Insurance Study (FIS) profiles. When current information and/or estimates of site conditions or flows differ significantly from adopted regulatory information (FIS), it may be necessary to compute both “design” and “regulatory” existing and proposed conditions.

There are a number of encompassing features of a steady state (flow is constant) hydraulic model for a roadway stream crossing. They include the natural adjacent floodplain, subject structure, any supplemental structures, and the roadway. Accurate modeling and calculations need to account for all potential conveyance mechanisms. Generally, most modern step-backwater methodologies can incorporate all of the above elements in the evaluation of hydraulic characteristics of the project site.

1) Bridge Hydraulics

The three most common types of flow through bridges are free surface flow (low flow), free surface (unsubmerged) orifice flow and submerged orifice flow. The latter two are also referred to as pressure flow. All of the above flow conditions may also occur simultaneously with flow over the roadway.

There are situations in which steep stream slopes are encountered and the flow may be supercritical (Froude No. > 1). This is a situation in which theoretically no backwater is created. For critical and supercritical flow situations the profile calculation would proceed from upstream to downstream. If this situation is encountered, the accuracy of the hydraulic model may be suspect and it is questionable whether the bridge should impose any constrictions on the stream channel. Sufficient clearance should be provided to insure that the superstructure will not come in contact with the flow.

Generally, in Wisconsin, most natural stream flow in a sub-critical (Froude No. < 1) regime. Therefore, the water surface profile calculation will proceed from downstream to upstream.

A sample problem of bridge hydraulic using WSPRO is given below:

Sample Problem

1. Computed Q_{100} = 750 c.f.s.

2. Slope = .004 '/' (survey 1500 feet up/down stream)

3. The cross-sections selected for the WSPRO analysis are shown on the contour map (Figure 8.3.1). The plotted sections with sub-areas and assigned "n" values are shown in Figures 8.3.2 and 8.3.3.

4. The proposed bridge is a 35 ft. concrete flat slab on A-1 abutments. From the output the Bridge area = 111.2 sq. ft., Bridge velocity = 6.75 ft./sec, Backwater = 0.654 ft., High-water elev. 972.267. See WSPRO Sample Problem Input and Output.

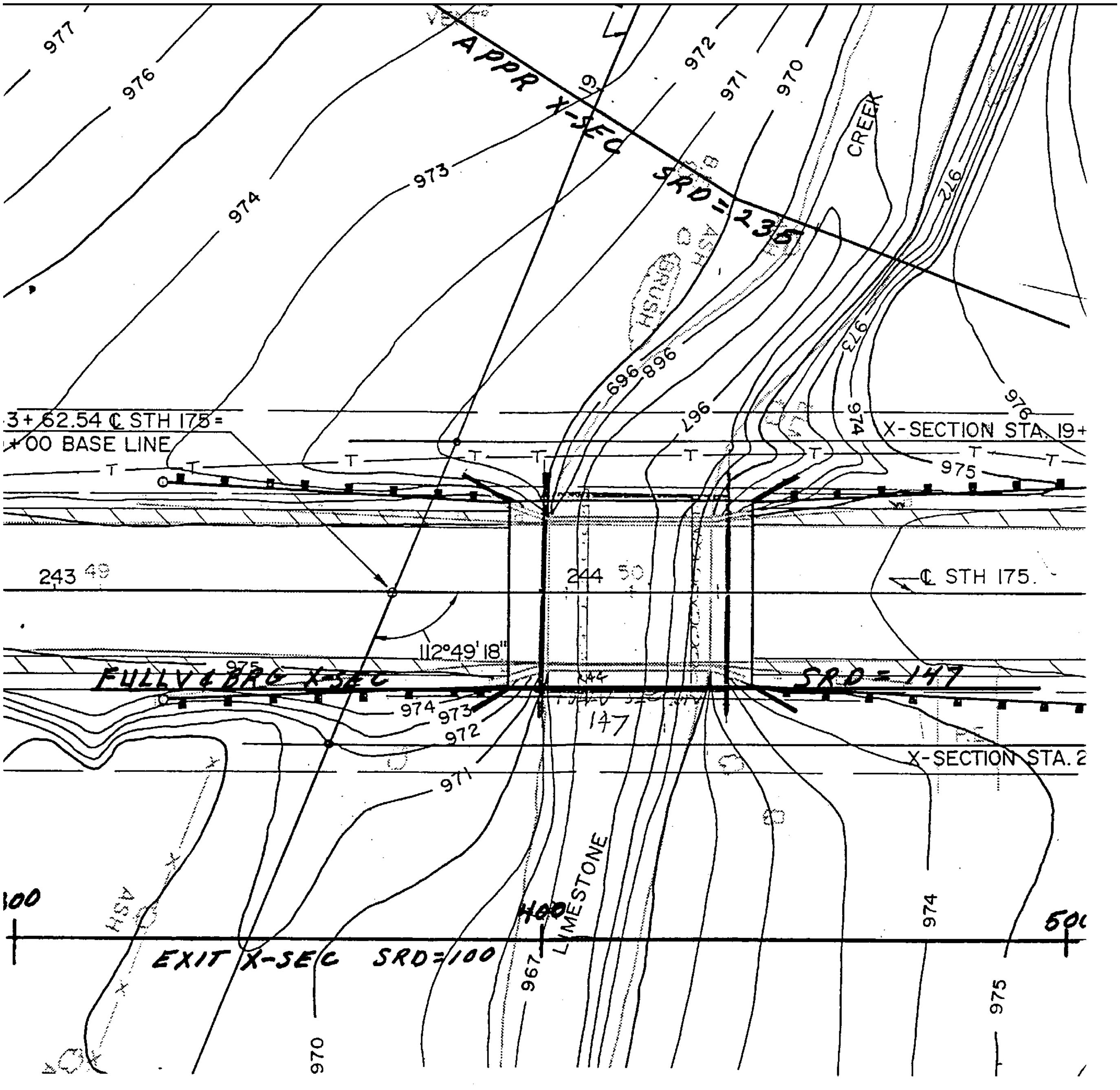


FIGURE 8.3.1
Contour and Cross-Section Location

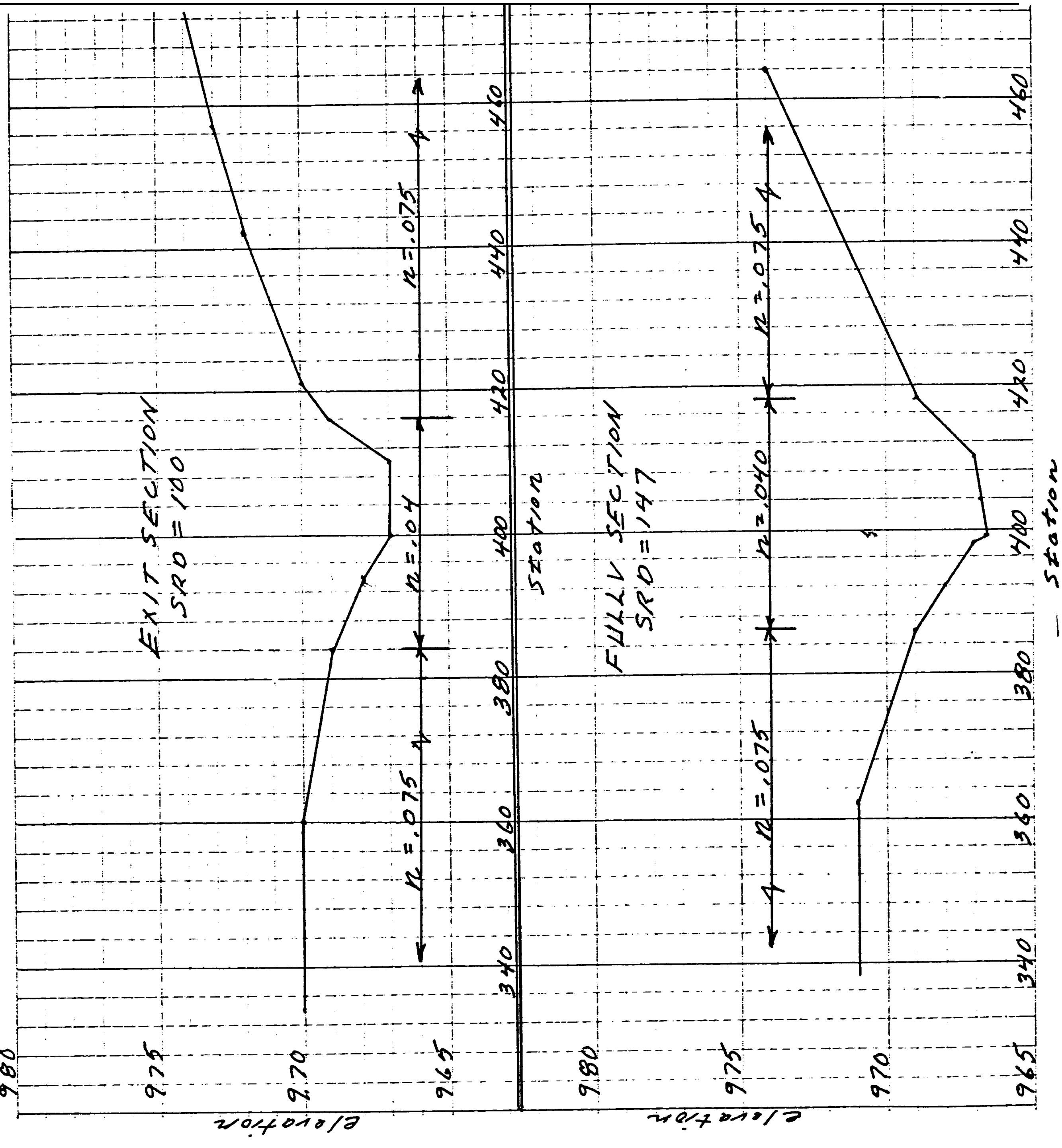


FIGURE 8.3.2
WSPRO CROSS-SECTIONS

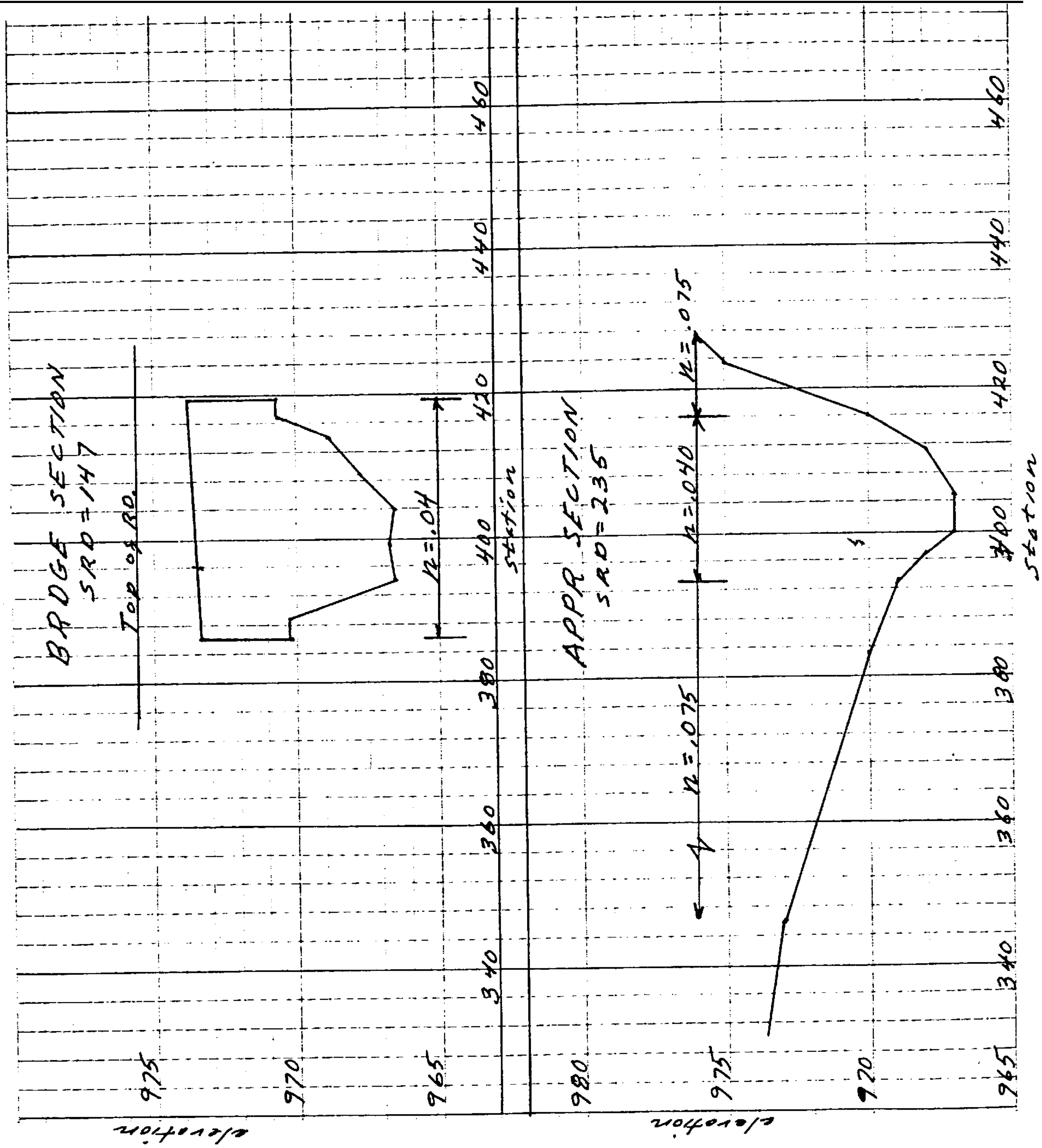


FIGURE 8.3.3
WSPRO CROSS-SECTIONS

```
T1      B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
T2      TOWN OF WINGPORT, MADISON COUNTY.
T3      PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99
*
*
J3      6,5,17,13,15,7,3
*
SI 0
*
Q       750
*
SK       0.004
*
*
XS   EXIT  000
*
GR       200,970 323,970 360,970 384,969 394,968
GR       400,967 410,967 416,969 421,970 442,972 457,973
GR       489,975 519,976
*
N       0.075, 0.040, 0.075
*
SA       384, 416
*
*
XS   FULV  147
*
GR       200,971 362,971 386,969 392,968 398,967 399,966.6
GR       404,966.7 410,967 418,969 462,974 519,976
*
N       0.075, 0.040, 0.075
*
SA       386, 418
*
*
BR   BRDG  147, 973.4
*
GR       386,973.4 386,970.2 388.6,970.2 394,966.6 399,966.7
GR       404,966.6 409,967.8 414,968.9 416.5,970.6 419,970.6
GR       419,973.8 386,973.4
*
CD       3   38   3.0  975.5
*
```

```
N        0.04
*

SA
*
*
XS   APPR  235
*
GR        251,978 301,975 319,974 346,973 383,970 393,969
GR        397,968 400,967 405,967 411,968 416,970 423,975
GR        427,976
*
N        0.075, 0.040, 0.075
*
SA        393, 416
*

EX
*

ER
```

```
    ************************* W S P R O ***************************
Federal Highway Administration  -  U. S. Geological Survey
    Model for Water-Surface Profile Computations.
       Run Date & Time:  3/ 9/1999  5:44 pm     Version V061698
         Input File: B75-99P.WSP    Output File: B75-99P.LST
    *-----------------------------------------------------------*
  T1        B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
  T2        TOWN OF WINGPORT, MADISON COUNTY.
  T3        PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99
  J3        6,5,17,13,15,7,3
+++082 NOTICE:  J3 Record Replaced With UT Record (See Users Manual).
 S10
Q           750

***   Processing Flow Data; Placing Information into Sequence  1   ***

  SK        0.004
************************* W S P R O ****************************************
        Federal Highway Administration  -  U. S. Geological Survey
         Model for Water-Surface Profile Computations.
         Input Units: English  /  Output Units: English
  *-----------------------------------------------------------------*
B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
         TOWN OF WINGPORT, MADISON COUNTY.
    PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99

    *-------------------------------------------------*
    *   Starting To Process Header Record EXIT      *
    *-------------------------------------------------*

  XS   EXIT  000
  GR        200,970 323,970 360,970 384,969 394,968
  GR        400,967 410,967 416,969 421,970 442,972 457,973
  GR        489,975 519,976
  N        0.075, 0.040, 0.075
  SA        384, 416

   ***   Completed Reading Data Associated With Header Record EXIT    ***
   ***  Storing X-Section Data In Temporary File As Record Number  1  ***

   ***            Data Summary For Header Record EXIT              ***
   SRD Location:       0.   Cross-Section Skew:    .0   Error Code   0
   Valley Slope:   .00000    Averaging Conveyance By Geometric Mean.
```

```
   Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                  X,Y-coordinates (13 pairs)
      X          Y            X          Y            X          Y
 ----------  ----------   ----------  ----------   ----------  ----------
    200.000   970.000    323.000    970.000    360.000     970.000
    384.000   969.000    394.000    968.000    400.000     967.000
    410.000   967.000    416.000    969.000    421.000     970.000
    442.000   972.000    457.000    973.000    489.000     975.000
    519.000   976.000
 ----------  -------------   ----------  -----------   ----------   ----------

                  Minimum and Maximum X,Y-coordinates
    Minimum X-Station:     200.000  ( associated Y-Elevation:  970.000 )
    Maximum X-Station:     519.000  ( associated Y-Elevation:  976.000 )
    Minimum Y-Elevation:   967.000  ( associated X-Station:    410.000 )
    Maximum Y-Elevation:   976.000  ( associated X-Station:    519.000 )


                  Roughness Data (  3 SubAreas )
                          Roughness    Horizontal
                  SubArea  Coefficient   Breakpoint
                  -------  -----------  ------------
                     1         .075          ---
                               ---       384.000
                     2         .040          ---
                               ---       416.000
                     3         .075          ---
                  -------  -----------  ------------


                  *-------------------------------------------------*
                  * Finished Processing Header Record EXIT        *
                  *-------------------------------------------------*
************************  W S P R O  ***********************************
    Federal Highway Administration  -  U. S. Geological Survey
         Model for Water-Surface Profile Computations.
       Input Units: English  /  Output Units: English
 *------------------------------------------------------------------------------------------*
    B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
              TOWN OF WINGPORT, MADISON COUNTY.
       PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99
```

```
     *--------------------------------------------------*
     *  Starting To Process Header Record FULV          *
     *--------------------------------------------------*

XS   FULV  147
GR       200,971 362,971 386,969 392,968 398,967 399,966.6
GR       404,966.7 410,967 418,969 462,974 519,976
N       0.075, 0.040, 0.075
SA       386, 418

 ***  Completed Reading Data Associated With Header Record FULV   ***
 ***  Storing X-Section Data In Temporary File As Record Number  2  ***

 ***            Data Summary For Header Record FULV             ***
 SRD Location:      147.   Cross-Section Skew:    .0   Error Code   0
 Valley Slope:   .00000    Averaging Conveyance By Geometric Mean.
 Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                X,Y-coordinates (11 pairs)

     X          Y            X          Y            X          Y
------------ ----------- ---------- ---------- ---------- ----------
   200.000    971.000    362.000    971.000    386.000    969.000
   392.000    968.000    398.000    967.000    399.000    966.600
   404.000    966.700    410.000    967.000    418.000    969.000
   462.000    974.000    519.000    976.000
------------ ----------- ---------- ---------- ---------- ----------

                Minimum and Maximum X,Y-coordinates

  Minimum X-Station:    200.000  ( associated Y-Elevation:  971.000 )
  Maximum X-Station:    519.000  ( associated Y-Elevation:  976.000 )
  Minimum Y-Elevation:  966.600  ( associated X-Station:    399.000 )
  Maximum Y-Elevation:  976.000  ( associated X-Station:    519.000 )

                Roughness Data (  3 SubAreas )
                        Roughness    Horizontal
                SubArea  Coefficient  Breakpoint
                ------- ----------- ------------
                   1        .075         ---
                            ---        386.000
                   2        .040         ---
                            ---        418.000
                   3        .075         ---
                ------- ----------- ------------
```

```
          *------------------------------------------------*
          * Finished Processing Header Record FULV      *
          *------------------------------------------------*
************************ W S P R O **************************
Federal Highway Administration  -  U. S. Geological Survey
    Model for Water-Surface Profile Computations.
   Input Units: English  /  Output Units: English
*-------------------------------------------------------------*
 B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
       TOWN OF WINGPORT, MADISON COUNTY.
    PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99

*-----------------------------------------------------*
*       Starting To Process Header Record BRDG       *
*-----------------------------------------------------*

  BR   BRDG  147, 973.4
  GR        386,973.4 386,970.2 388.6,970.2 394,966.6 399,966.7
  GR        404,966.6 409,967.8 414,968.9 416.5,970.6 419,970.6
  GR        419,973.8 386,973.4
  CD        3   38   3.0  975.5
  N        0.04
   SA
  ***     Completed Reading Data Associated With Header Record BRDG     ***
+++072 NOTICE:  X-coordinate # 2 increased to eliminate vertical segment.
+++072 NOTICE:  X-coordinate #11 increased to eliminate vertical segment.
  ***     Storing Bridge Data In Temporary File As Record Number  3    ***

   ***            Data Summary For Bridge Record BRDG              ***
   SRD Location:      147.   Cross-Section Skew:    .0   Error Code   0
   Valley Slope:  *******    Averaging Conveyance By Geometric Mean.
   Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

               X,Y-coordinates (12 pairs)

     X         Y           X         Y          X        Y
  ----------  ----------   ---------  ---------  ---------  ---------
    386.000    973.400    386.100    970.200    388.600    970.200
    394.000    966.600    399.000    966.700    404.000    966.600
    409.000    967.800    414.000    968.900    416.500    970.600
    419.000    970.600    419.100    973.800    386.000    973.400
  ----------  ---------   ---------  ---------  ---------  ---------
```

```
                Minimum and Maximum X,Y-coordinates
   Minimum X-Station:     386.000  ( associated Y-Elevation:  973.400 )
   Maximum X-Station:     419.100  ( associated Y-Elevation:  973.800 )
   Minimum Y-Elevation:   966.600  ( associated X-Station:    404.000 )
   Maximum Y-Elevation:   973.800  ( associated X-Station:    419.100 )

                Roughness Data (  1 SubAreas )
                      Roughness    Horizontal
                SubArea  Coefficient  Breakpoint
                -------  -----------  ------------
                   1        .040        ---
                -------  -----------  ------------

                Discharge coefficient parameters
              BRType   BRWdth   EMBSS   EMBElv    UserCD
               3       38.000   3.00   975.500 **********

                  Pressure flow elevations
                    AVBCEL     PFElev
                  ************   973.400

                Abutment Parameters
   ABSLPL  ABSLPR  XTOELT   YTOELT   XTOERT   YTOERT
   *********  *********   ********* ************ ************  *************

              ** No Pier/Pile Data Encountered **

            *-----------------------------------------------*
           *   Finished Processing Header Record BRDG      *
           *-----------------------------------------------*
************************ W S P R O **************************

   Federal Highway Administration  -  U. S. Geological Survey
        Model for Water-Surface Profile Computations.
        Input Units: English  /  Output Units: English
  *-----------------------------------------------------------*
    B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
              TOWN OF WINGPORT, MADISON COUNTY.
        PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99

      *-----------------------------------------------------------*
       *    Starting To Process Header Record APPR      *
      *-----------------------------------------------------------*
```

```
XS   APPR  235
GR        251,978 301,975 319,974 346,973 383,970 393,969
GR        397,968 400,967 405,967 411,968 416,970 423,975
GR        427,976
N        0.075, 0.040, 0.075
SA        393, 416

 ***   Completed Reading Data Associated With Header Record APPR    ***
 ***  Storing X-Section Data In Temporary File As Record Number  4  ***

 ***             Data Summary For Header Record APPR              ***
 SRD Location:      235.   Cross-Section Skew:    .0   Error Code   0
 Valley Slope:   .00000    Averaging Conveyance By Geometric Mean.
 Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                X,Y-coordinates (13 pairs)

    X        Y          X        Y          X        Y
----------- ------------ ----------- ---------- ----------- ------------
   251.000    978.000     301.000    975.000     319.000    974.000
   346.000    973.000     383.000    970.000     393.000    969.000
   397.000    968.000     400.000    967.000     405.000    967.000
   411.000    968.000     416.000    970.000     423.000    975.000
   427.000    976.000
------------ ----------- ----------- ---------- ---------- ------------

                Minimum and Maximum X,Y-coordinates

  Minimum X-Station:     251.000  ( associated Y-Elevation:  978.000 )
  Maximum X-Station:     427.000  ( associated Y-Elevation:  976.000 )
  Minimum Y-Elevation:   967.000  ( associated X-Station:    405.000 )
  Maximum Y-Elevation:   978.000  ( associated X-Station:    251.000 )

                 Roughness Data (  3 SubAreas )
                       Roughness    Horizontal
                SubArea  Coefficient   Breakpoint
                ------- -----------    ------------
                   1        .075          ---
                           ---         393.000
                   2        .040          ---
                           ---         416.000
                   3        .075          ---
                ------- ----------- ------------
```

Bridge datum projection(s): XREFLT XREFRT FDSTLT FDSTRT

```
                  ******* ******* ******* *******

          *-----------------------------------------------*
          * Finished Processing Header Record APPR      *
          *-----------------------------------------------*
************************ W S P R O **************************
         Federal Highway Administration  -  U. S. Geological Survey
             Model for Water-Surface Profile Computations.
               Input Units: English  /  Output Units: English
     *----------------------------------------------------------------------*
       B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
                TOWN OF WINGPORT, MADISON COUNTY.
               PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99

EX

         *=====================================================*
         *        Summary of Boundary Condition Information    *
         *=====================================================*

            Reach      Water Surface   Friction
      #   Discharge      Elevation      Slope        Flow Regime
     --   ---------    -------------   --------   --------------------
      1     750.00       ********         .0040      Sub-Critical
     --   ---------    -------------   --------   --------------------
```

```
        *=======================================================*
        *       Beginning  1 Profile Calculation(s)       *
        *=======================================================*
************************  W S P R O  **************************
        Federal Highway Administration  -  U. S. Geological Survey
            Model for Water-Surface Profile Computations.
            Input Units: English  /  Output Units: English
    *------------------------------------------------------------*
        B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
                TOWN OF WINGPORT, MADISON COUNTY.
            PROPOSED CONDITIONS    BY WLO OF WISDOT 3-8-99

                << Beginning Computations for Profile  1 >>

                WSEL    VHD      Q        AREA      SRDL      LEW
                EGEL     HF      V         K       FLEN      REW
                CRWS     HO     FR #       SF      ALPHA      ERR
              --------- ------ ---------- ---------- --------- ---------
 Section: EXIT     970.925   .275   750.000   295.364 *********   200.000
 Header Type: XS    971.200 ******      2.539   11856.03 *********   430.710
 SRD:      .000    970.576 ******      .656     ******     2.747     ******

===125 FR# EXCEEDS FNTEST AT SECID “FULV “:  TRIALS CONTINUED.
      FNTEST, FR#, WSEL, CRWS:  .80     .81     971.43     970.20

===110 WSEL NOT FOUND AT SECID “FULV “:  REDUCED DELTAY.
      WSLIM1, WSLIM2, DELTAY:  970.20     976.00     .50

===115 WSEL NOT FOUND AT SECID “FULV “:  USED WSMIN = CRWS.
      WSLIM1, WSLIM2, CRWS:  970.20     976.00     970.20

 Section: FULV     971.419   .355    750.000    252.007    147.000   200.000
 Header Type: FV    971.774   .528      2.976   13219.32   147.000   439.290
 SRD:    147.000    970.200   .040      .821      .0036     2.576      .007

    <<< The Preceding Data Reflect The “Unconstricted” Profile >>>

===125 FR# EXCEEDS FNTEST AT SECID “APPR “:  TRIALS CONTINUED.
     FNTEST, FR#, WSEL, CRWS:  .80     .88     971.60     971.14


===110 WSEL NOT FOUND AT SECID “APPR “:  REDUCED DELTAY.
     WSLIM1, WSLIM2, DELTAY:  971.14     978.00     .50
```

```
===115 WSEL NOT FOUND AT SECID "APPR ": USED WSMIN = CRWS.
    WSLIM1, WSLIM2, CRWS:  971.14   978.00   971.14

===135 CONVEYANCE RATIO OUTSIDE OF RECOMMENDED LIMITS AT SECID "APPR ".
    KRATIO:  .63

 Section: APPR     971.613  .863   750.000   124.591   88.000  363.107
 Header Type: AS    972.476  .448     6.020   8351.22   88.000  418.258
 SRD:   235.000    971.137  .254      .874     .0051    1.531    -.001

   <<< The Preceding Data Reflect The "Unconstricted" Profile >>>
   <<< The Following Data Reflect The "Constricted" Profile >>>
   <<< Beginning Bridge/Culvert Hydraulic Computations >>>
                 WSEL   VHD     Q       AREA     SRDL     LEW
                 EGEL    HF      V        K       FLEN     REW
                 CRWS    HO     FR #      SF      ALPHA    ERR
               --------- ------ ---------- ---------- --------- ---------
 Section: BRDG     971.462  .708   750.000   111.160  147.000  386.061
 Header Type: BR    972.170  .807     6.747   8613.35  147.000  419.027
 SRD:   147.000    970.540  .160      .648    ******    1.000    -.007

 Specific Bridge Information  C    P/A  PFELEV   BLEN    XLAB    XRAB
 Bridge Type 3  Flow Type 1 ------ ----- -------- -------- --------- ---------
 Pier/Pile Code **        1.0000  .000  973.400 ******** ********* *********
 -------------------------- ------ ----- -------- -------- --------- ---------
   Unconstricted Full Valley Section Water Surface Elevation:   971.419
   Downstream Bridge Section Water Surface Elevation:            971.462
   Bridge DrawDown Distance:                                      -.043
 ---------------------------------------------------------------------------

                 WSEL   VHD     Q       AREA     SRDL     LEW
                 EGEL    HF      V        K       FLEN     REW
                 CRWS    HO     FR #      SF      ALPHA    ERR
               --------- ------ ---------- ---------- --------- ---------
 Section: APPR     972.267  .548   750.000   163.600   50.000  355.041
 Header Type: AS    972.815  .285     4.584  11437.87   50.325  419.174
 SRD:   235.000    971.137  .359      .655     .0051    1.677    -.008

      ** Change in Approach Section Water Surface Elevation:  .654 **

          Approach Section APPR  Flow Contraction Information
```

M(G)	M(K)	KQ	XLKQ	XRKQ	OTEL
.402	.080	10562.0	387.546	420.512	972.267

<<< End of Bridge Hydraulics Computations >>>

<< Completed Computations of Profile 1 >>

************************* W S P R O ***

Federal Highway Administration - U. S. Geological Survey
Model for Water-Surface Profile Computations.
Input Units: English / Output Units: English

--

B-75-99, STH 175, OVER LIMESTONE CREEK I.D. #8510-15-00
TOWN OF WINGPORT, MADISON COUNTY.
PROPOSED CONDITIONS BY WLO OF WISDOT 3-8-99

=== User Defined Table 1 of 1 ===

	SRD	Q	AREA	VEL	CRWS	EGL	WSEL
1 EXIT	.000	750.000	295.4	2.539	970.576	971.200	970.925
2 FULV	147.000	750.000	252.0	2.976	970.200	971.774	971.419
3 APPR	235.000	750.000	124.6	6.020	971.137	972.476	**971.613**
4 BRDG	147.000	750.000	111.2	6.747	970.540	972.170	971.462
5 APPR	235.000	750.000	163.6	4.584	971.137	972.815	**972.267**

ER

****************** Normal end of WSPRO execution. *****************

*************** Elapsed Time: 0 Minutes 1 Seconds **************

******* **NOTE:** ***Backwater = 972.267 - 971.613 = 0.654 feet*** ********

2) Roadway Overflow

One potential element in developing a hydraulic model for a stream crossing is roadway overflow. It is sometimes necessary to compute flow over highway embankments in combination with flow through structure openings. Most automated methodologies will incorporate the division of flow through a structure and over the road in determination of the solution. The WSPRO methodology will conduct the “combined flow” solution and internally determine and adjust the coefficient of discharge for both the structure and roadway weir section. Other methodologies (i.e. HEC-2, HEC-RAS) rely on user defined coefficients for both the structure and roadway flow solutions. The discharge equation and coefficients for flow over a highway embankment are given in this section.

The geometry and flow pattern for a highway embankment are illustrated in Figure 8.3.4. Under free flow conditions critical depths occur near the crown line. The head (H) is referred to the elevation of the water above the crown, and the length (L), in direction of flow, is the distance between the points of the upstream and downstream embankment faces (edge of shoulder). The length (B) of the embankment has no influence on the discharge coefficient.

The discharge equation is $Q = k_t C B H^{3/2}$

Q = discharge
C = coefficient of discharge
B = length of flow section along the road normal to the direction of flow
H = total head = $h + h_v$
k_t = submergence factor

The length of overflow section (B) will be a function of the roadway profile grade line and depth of over-topping (h). Coefficient (C) is obtained by computing h/L and using Chart 8.3.1 or Chart 8.3.2, for paved or gravel roads.

The degree of submergence of a highway embankment is defined by ration ht/H. The effect of submergence on the discharge coefficient (C) is expressed by the factor kt as shown in Chart 8.3.3. The factor kt is multiplied by the discharge coefficient (C) for free-flow conditions to obtain the discharge coefficient for submerged conditions. If the degree of submergence is greater than 0.9, the computed discharge may not be reliable. However, the portion of the total flow which passes over the road as compared to that which goes through the bridge is usually small, and thus a greater error can be tolerated in this computation.

Further discussion for road overflow is found in reference (4).

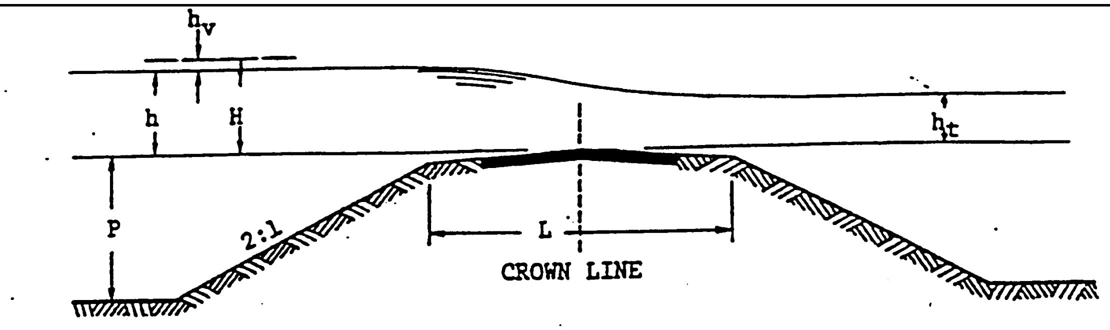


FIGURE 8.3.4 – DEFINITION SKETCH OF FLOW OVER A HIGHWAY EMBANKMENT

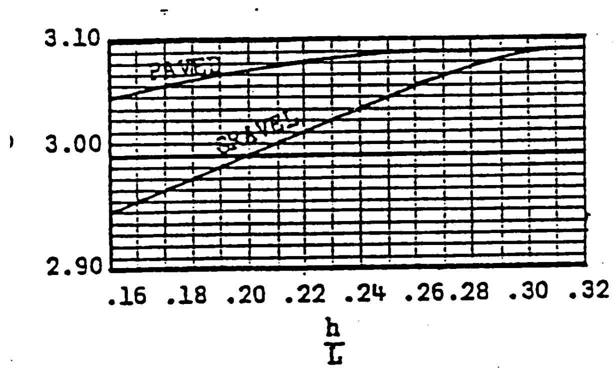


CHART 8.3.1 - DISCHARGE COEFFICIENTS FOR HIGHWAY EMBANKMENTS FOR h/L RATIOS > 0.15

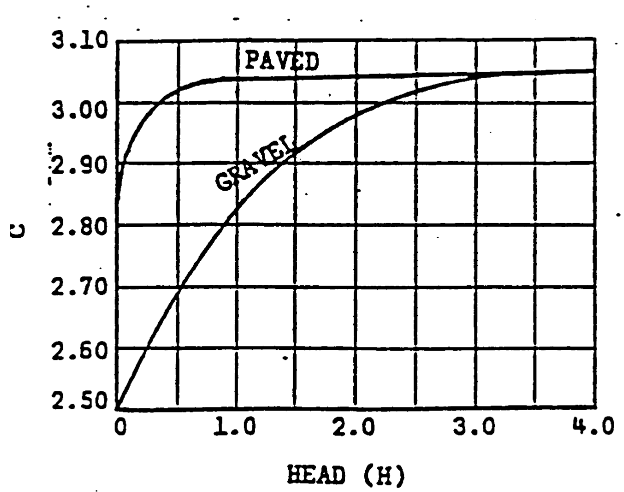


CHART 8.3.2 – DISCHARGE COEFFICIENTS FOR HIGHWAY EMBANKMENTS FOR H/L RATIOS < 0.15

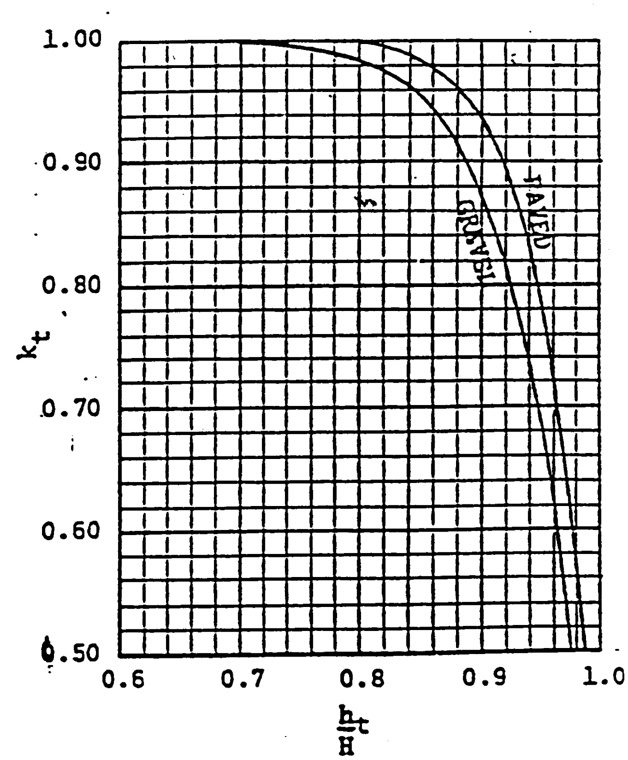


CHART 8.3.3 - DEFINITION OF ADJUSTMENT FACTOR, k_t, FOR SUBMERGED HIGHWAY EMBANKMENTS

G. Conduct Scour Evaluation

Evaluating scour potential at bridges is based on recommendations and background from FHWA Technical Advisory “Evaluating Scour at Bridges” dated October 28, 1991 and procedures from the FHWA Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges revised April 1993 (English), November 1995 (Metric), and Hydraulic Engineering Circular No. 20 Stream Stability at Highway Structures, February 1991 (English), November 1995 (Metric).

All bridges shall be evaluated to determine the vulnerability to scour. In the FHWA publication Recording and Coding Guide for Structure Inventory and Appraisal of the Nation’s Bridges (9), a code system has been established for evaluation. A section in this guide “Item 113 - Scour Critical Bridges” uses a single-digit code to identify the status of the bridge regarding its vulnerability to scour. A reproduction of this item is given in Appendix 8-D.

There are three main components of total scour at a bridge site. They are Long-term Aggradation and Degradation, Contraction Scour, and Local Scour. In addition, lateral migration of the stream must be assessed when evaluating total scour at substructure units. Contraction and local scour will be evaluated in the context of clear-water and live bed scour conditions.

Live Bed and Clear Water Scour

Clear-water scour occurs when there is no or insignificant movement (transport) of the bed material by the flow upstream of the crossing, but the acceleration of flow and vortices created by the piers or abutments causes the bed material in the vicinity of the crossing to move.

Live-bed scour occurs when there is significant transport of bed material from the upstream reach into the crossing.

1) Long-term Aggradation and Degradation

Aggradation is the deposition of eroded material in the stream from the upstream watershed. Degradation is the scouring (removal) of the streambed resulting from a deficient supply of sediment. These are subtle long term streambed elevation changes. These processes are natural in most cases. However, unnatural changes like dam construction or removal, as well as urbanization may cause Aggradation and Degradation. Excellent reference on this subject and the geomorphology streams is Highways in the River Environment (7), HEC-18, Evaluating Scour at Bridges (6), and HEC-20, Stream Stability at Highway Structures (10).

2) Contraction Scour

Generally, Contraction scour is caused by bridge approaches encroaching onto the floodplain and decreasing the flow area resulting in an increase in velocity through a bridge opening. The higher velocities are able to transport sediment out of the contracted area until an equilibrium is reached. Contraction scour can also be caused by short term changes in the downstream water surface elevation, such as bridges located on a meander bend or bridges located in the backwater of dams with highly fluctuating water levels. See reference (6)&(10) for discussion and methods of analysis.

Computing Contraction Scour.

Live-Bed Contraction Scour

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad \text{(English \& Metric)}$$

$$y_s = y_2 - y_0 = \ (average\ scour\ depth)$$

y_s = Average scour depth, ft, m
y_1 = Average depth in the upstream main Channel, ft, m
y_2 = Average depth in the contracted section, ft, m
y_0 = Existing depth in the contracted section before scouring, ft, m
Q_1 = Flow in upstream channel transporting sediment, ft^3/s, m^3/s
Q_2 = Flow in contracted channel, ft^3/s, m^3/s
W_1 = Bottom Width of upstream main channel, ft, m
W_2 = Net bottom Width of channel at contracted section, ft, m
k_1 = Exponent for mode of bed material transport, (0.59 - 0.69 see ref. 6))

Clear-Water Contraction Scour

$$y_2 = \left[\frac{Q^2}{120\ D_m^{\frac{2}{3}} W^2}\right]^{\frac{3}{7}} \qquad \text{(English)}$$

$$y_2 = \left[\frac{0.025Q^2}{D_m^{\frac{2}{3}} W^2}\right]^{\frac{3}{7}} \quad \text{(Metric)}$$

$$y_s = y_2 - y_0 = \ (average\ scour\ depth)$$

y_s = Average scour depth, ft, m
y_2 = Average depth in the contracted section, ft, m
y_0 = Existing depth in the contracted section before scouring, ft, m
Q = Discharge through the bridge associated with W, ft^3/s, m^3/s
D_m = Diameter of the smallest nontransportable particle ($1.25D_{50}$), ft, m
D_{50} = Median Diameter of the bed material, 50% smaller than ft, m
W = Net bottom Width of channel at contracted section, ft, m

3) Local Scour

Local scour is the removal of material from around a pier abutment, spur dike, or the embankment. It is caused by an acceleration of the flow and/or resulting vortices induced by obstructions to flow.

Pier Scour & Colorado State University's (CSU) Equation

The recommended equation for determination of pier scour in reference (6) is the CSU's equation. Velocity is a factor in calculating the Froude Number. Therefore it is applicable where a hydraulic model of the bridge is available. The equation and appropriate charts and tables are shown below in Figure 8.3.5. See reference (6) for a complete discussion of the CSU Equation.

Computing Pier Scour. The CSU equation for pier scour is:

$$\frac{y_s}{a} = 2.0\ K_1\ K_2\ K_3 K_4 \left(\frac{y_1}{a}\right)^{0.35} \quad Fr_1^{0.43} \quad \text{(English \& Metric)}$$

y_s = Scour depth, ft, m
y_1 = Flow depth directly upstream of the pier, ft, m
a = Pier width, ft, m
Fr_1 = Froude number directly upstream of the pier = $V_1/(gy_1)$
V_1 = Mea Velocity of flow directly upstream of the pier, ft/s, m/s
g = Acceleration of gravity, 32.2 ft/s^2, 9.81 m/s^2

K_1 = Correction Factor for pier nose shape (see Table 2 of reference 6)
K_2 = Correction Factor for angle of attack of flow (see Table 3 of reference 6)
K_3 = Correction Factor for bed condition (see Table 4 of reference 6)
K_4 = Correction Factor for armoring by bed material 0.7 - 1.0 (see reference 6)

Table 2. Correction Factor, K_1, for Pier Nose Shape.	
Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 3. Correction Factor, K_2, for Angle of Attack, θ, of the Flow.			
Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow
L = length of pier, m

Table 4. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.		
Bed Condition	Dune Height m	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$3 > H \geq 0.6$	1.1
Medium Dunes	$9 > H \geq 3$	1.2 to 1.1
Large Dunes	$H \geq 9$	1.3

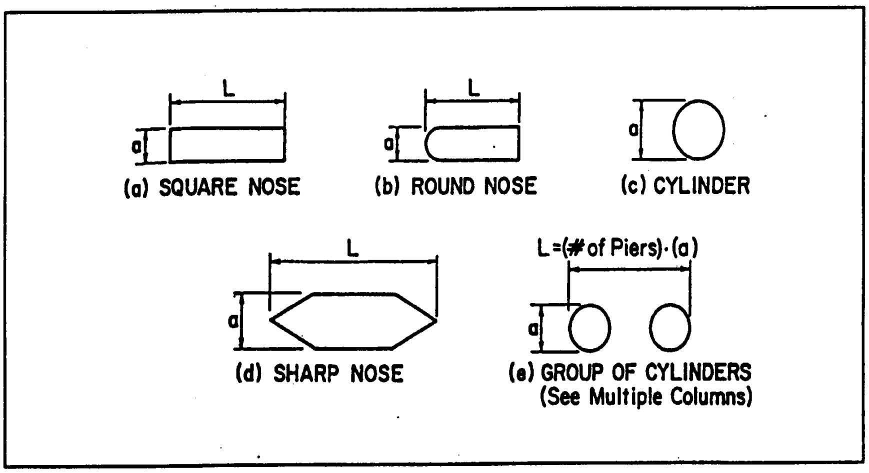


Figure 8.3.5

Abutment Scour

Abutment Scour Equations

Abutment scour analysis is dependent on equations that relate to the degree of projection of encroachment (embankment) into the flood plain. Lack of field data to verify any one equation causes doubt on the reliability of scour estimates. This is one of the reasons heavy riprap underlain with geotextile fabric is used to resist scour as described in the construction specifications at all stream crossing abutments.

Froelich's Live-Bed Scour at Abutments

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \quad \text{(English \& Metric)}$$

y_s = Scour depth, ft, m
y_a = Average depth of flow on the floodplain, ft, m
L' = Length of abutment(embankment) projected normal to flow, ft, m
Fr = Froude number of approach flow upstream of the abutment = $V_e/(gy_a)$
V_e = Q_e/A_e, ft/s, m/s
g = Acceleration of gravity, 32.2 ft/s^2, 9.81 m/s^2
A_e = Flow Area of approach cross section obstructed by embankment, ft^2, m^2
Q_e = Flow obstructed by abutment or approach embankment, ft^3/s, m^3/s
K_1 = Coefficient for abutment shape (see Table 6 of reference 6)
K_2 = Coefficient for angle of embankment to flow (see Figure 16 of reference 6)

The HIRE Equation for Live-Bed Scour at Abutments

$$\frac{y_s}{y_1} = 4\, Fr_1^{0.33} \frac{K_1}{0.55} \quad \text{(English \& Metric)}$$

y_s = Scour depth, ft, m
y_1 = Depth of flow at the abutment on the overbank or in the main channel, ft, m
K_1 = Coefficient for abutment shape (see Table 6 of reference 6)

Fr = Froude number based on velocity and depth adjacent to and upstream of the abutment

Abutment Shape Coefficients (k_1)

Description	k_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

The above equations often predict excessively conservative abutment scour depths. This results from there development in laboratory flume experiments that did not reflect the typical geometry or flow distribution associated with roadway encroachments of floodplains.

4) Design Considerations for Scour

Provide adequate free board (2 feet desirable) for when possible to prevent pressure flow occurrences.

Pier foundations elevations on floodplains should be designed considering the potential of channel or thalweg migration over the design life of the structure.

Align all substructure units and especially pier with the direction of flow. Improper alignment may significantly increase the magnitude of scour.

Piers in the water should have a rounded or streamline nose to reduce turbulence and related scour potential.

Spill-through (sloping) are less vulnerable to scour than vertical wall abutments.

Current equations (6) and methods used to estimate the magnitude of abutment scour were developed in a laboratory under ideal conditions and lack adequate field verification. These equations may tend to over estimate the magnitude of scour. These equations should be incorporated with a great deal of discretion.

H. Select Bridge Design Alternatives

In most design situations, the “proposed bridge” design will be based on the several pertinent design factors discussed in Section 8.4.(1). They will dictate the final selection of bridge length, abutment design, superstructure design and approach roadway design. The Hydraulic/Site report should adequately document the site characteristics, hydrologic, hydraulic, results, as well as the alternatives considered.

See Appendix 8-A for a sample Hydraulic/Site Report.

(3) References

1) Ven Te Chow, Ph.D. Open Channel Hydraulics (New York, McGraw-Hill Book Company 1959).

2) U.S. Department of Transportation, Federal Highway Administration, Design Charts for Open-Channel Flow Hydraulic Design Series No. 3, August 1961.

3) U.S. Department of Interior, Geological Survey, Measurement of Peak Discharge at Width Contractions by Indirect Methods; Techniques of Water-Resources Investigation of the U.S.G.S., Chapter A4, Book 3, Third printing 1976.

4) J. O. Shearman, W. H. Hirby, V. R. Schneider, H. N. Flippo, Bridge Waterways Analysis Model: Research Report, Federal Highways Administration Report No. FHWA/RD-86/108.

5) U.S. Army Corps of Engineers, HEC2 Water Surface Profiles Users Manual Computer Program 723-X6-L202A Hydrologic Engineering Center, Davis CA. Revised February 1991.

5a) U.S. Army Corps of Engineers, HEC-RAS River Analysis System Users Manual, (CPD-68), Hydrologic Engineering Center, Davis CA. Version 2.0 April 1997.

6) U.S Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges, revised (April 1993 English), (November 1995 Metric).

7) U.S. Department of Transportation, Federal Highway Administration, Highways in the River Environment, February 1990.

8) Emmett M. Laursen and Arthur Toch, Scour Around Bridge Piers and Abutments, Iowa Institute of Hydraulic Research Bulletin No. 4, Iowa Highway Research Board May 1956.

9) U.S. Department of Transportation, Federal Highway Administration, Recording and Coding Guide for the Structure Inventory and Appraisal of the

Nation's Bridges, Office of Engineering, Bridge Division, Report No. FHWA-ED-89-044, December 1988.

10) U.S Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures, (February 1993 English), (November 1995 Metric).

8.4 HYDRAULIC DESIGN OF BOX CULVERTS

Box culverts are an efficient and economical design alternative for roadway stream crossings in the 300 to 1500 cfs discharge range. As a general guide culvert pipes are best suited for smaller discharge values while bridges are better suited for larger values. Although multicell box culverts are designed for larger discharges, the larger size culverts tend to lose the hydraulic and economic advantage over bridges. The following subsections discuss the design considerations and hydraulic design procedures for box culverts.

(1) Hydraulic Design Factors As in the hydraulic design of bridges, several hydraulic factors dictate the design of both the culvert and approach roadway. The critical hydraulic factors for design considerations are:

A. Economics

The best economics for box culvert design are realized with the culvert flowing full and producing a reasonable headwater depth (HW) within the boundary of other hydraulic and roadway design constraints.

For long box culverts, particularly on steep slopes, a considerable savings can be realized by incorporating an improved inlet design known as "Tapered Inlets". The improved efficiency of the inlet where the inlet controls the headwater, will allow for design of a smaller culvert barrel. See reference (1) for discussion on "Tapered Inlets".

B. Minimum Size

If the highway grade permits, a minimum five foot box culvert height is desirable for clean-out purposes.

C. Allowable Velocities and Outlet Scour

Generally, for velocities under 10 fps no riprap is needed at the discharge end of a box culvert, although close examination of local soil conditions is advisable.

For outlet velocities from 10-14 fps heavy riprap shall be used extending 15 to 35 feet from the end of the culvert apron.

For velocities greater than 14 fps energy dissipators should be considered. These are the most expensive means of end protection. See Section 8.4(G) for the hydraulic design of energy dissipators.

When heavy riprap is used it is carried up the slopes around the ends of the outlet apron to an elevation at mid-length of apron wing.

D. Roadway Overflow

See Section 8.3(1)B.

E. Culvert Skew

See Section 8.3(1)C.

F. Backwater and Highwater Elevations

The “Highwater elevation” commonly referred to as headwater for culverts, is the backwater created at the upstream end of the culvert. Although culverts are more hydraulically efficient and economical when flowing under a reasonable headwater, several factors shall be considered in determining an allowable highwater elevation. For further discussion see Section 8.3(1)D.

G. Debris Protection

Debris protection is provided where physical study of the drainage area indicates considerable debris collection. Debris protection structural design is a part of the culvert design, where used. A part of the box culvert survey report must justify the need for protection. Sample debris protection devices are shown in an older Bureau of Public Roads publication, *Hydraulic Engineering Circular No. 9* (2).

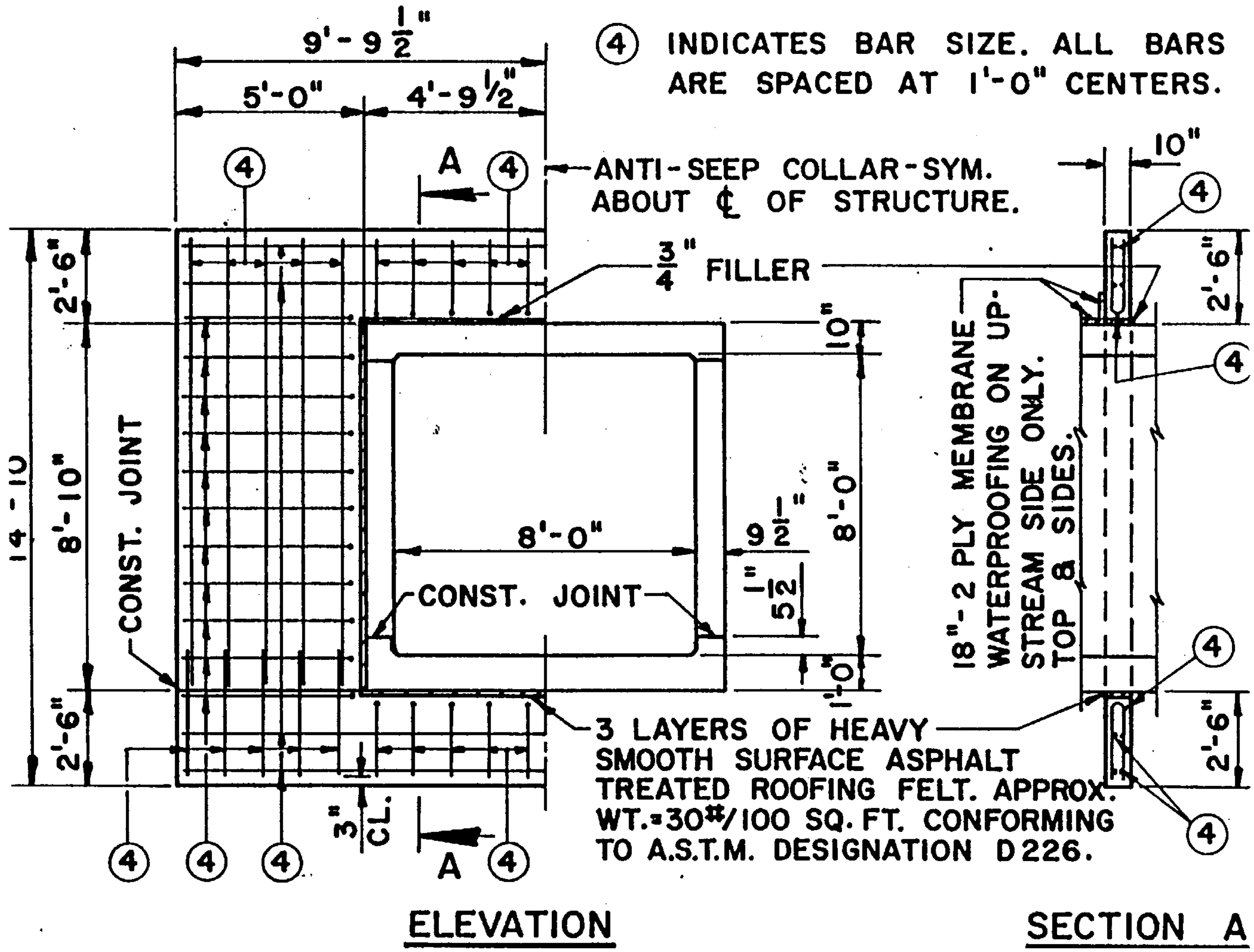
H. Anti-Seepage Collar

Anti-seepage collars are used to prevent the movement of water along the outside of the culvert and the failure by piping of the fill next to the culvert. They are used in sandy fills where the culvert has a large headwater.

Collars are located at the midpoint and upstream quarter point on long box culverts. If only one collar is used, it is located far enough from the inlet to intercept the phreatic (zero pressure) line to prevent seepage over the top of the collar. See reference (3).

A typical collar is shown in Figure 8.4.1 and is applicable to all single and twin box structures.

An alternate method of preventing seepage is to use a minimum one foot thick impervious soil blanket around the culvert inlet extending five feet over undisturbed embankment. The same effect can be obtained by designing seepage protection into the endwalls.



ANTI – SEEPAGE COLLAR

FIGURE 8.4.1

I. Weep Holes

The need for weep holes should be investigated for clay type soils with high fills, and should be eliminated in other cases.

If weep holes are necessary, alternate layers of fine and coarse aggregate are placed around the holes starting with coarse aggregate next to the hole.

(2) Design Procedure

A. Determine Design Discharge

See Section 8.2 for procedures.

B. Determine Hydraulic Stream Slope

Section 8.3(2)B for procedures.

C. Determine Tailwater Elevation

The tailwater elevation is the depth of water in the natural channel computed at the outlet of the culvert. In situations of steeper slopes and small culverts, the tailwater is not a critical design factor. However, for mild slopes and larger culverts, the tailwater is a critical design factor. It may control the outlet velocity and depth of flow in the culvert.

The tailwater elevation is calculated using a typical section downstream of the outlet and performing a "normal depth" analysis. An older publication from the former Bureau of Public Roads entitled *Design Charts for Open Channels* (4) can be used to determine tailwater depth directly from the charts for symmetrical channel. Several other hydraulic engineering textbooks and handbooks discuss the method to calculate "normal depth" if an irregular cross-section is encountered in the downstream channel.

D. Design Methodology

The most prevalent design methodology for culverts is the procedure in the FHWA publication entitled *Hydraulic Design of Highway Culverts* Hydraulic Design series (HDS) No. 5 (5). It is highly recommended the designer should first thoroughly study the methodologies presented in this publication.

Several personal computer software programs are available from public and private sources which use the same technique and methodology presented in HDS No. 5. One public domain computer program developed by FHWA entitled *"HY8"* is based on the HDS No. 5 manual. This program and documentation are available through the "McTrans Center" at the University of Florida, Gainesville (see Appendix 8-C). WSPRO (see Section 8.3(2)E) has a culvert option using the same methodology. These same programs have the capability of allowing the user to calculate the tailwater based on a downstream section and to calculate a combination of culvert and roadway overflow.

E. Develop Hydraulic Model

There are two major types of culvert flow: (1) flow with inlet control, and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area, and the inlet geometry at the entrance are of primary importance. Outlet control involves the consideration of the tailwater in the outlet channel, the culvert slope, the culvert roughness, and the length of the culvert barrel, as well as inlet geometry and cross-sectional area.

Another design of Inlet control which is used frequently is "Tapered Inlets" or improved inlets. The slope-tapered and side-tapered inlets are more efficient hydraulically, and can be a more economical design for long culverts in flow with inlet control.

In all culvert design, headwater depth (HW) or depth of water at the entrance to a culvert is an important factor in culvert capacity. The headwater depth is the vertical height from the culvert invert elevation at the entrance to the total energy elevation of the headwater pool (depth plus velocity head). Because of the low velocities at the entrance in most cases and difficulty in determining the velocity head for all flows, the water surface elevation and the total energy elevation at the entrance are assumed to be coincident.

The box culvert charts presented here are inlet and outlet control nomograph Charts 8.4.1 and 8.4.2, and a critical depth Chart 8.4.3. Note the “Inlet Type” over the HW/D scales on Chart 8.4.1 and entrance loss coefficients “Ke” for inlet types on Chart 8.4.2. The following illustrative problems are examples of their use. Forms similar to Figure 8.4.2 are used for computation.

<u>Outlet Control Problem</u>. The information necessary to solve this problem is given in Figure 8.4.2.

Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D=1.08 from Chart 8.4.1.
<u>The HW = 1.08 (5 ft) = 5.4 ft.</u>

Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 180 ft. and type “C” inlet; H = 1.5 ft. from Chart 8.4.2, TW = 5.2 ft. = ho
<u>Then HW = H + Tho = Lso = 1.5 ft. + 5.2 ft. - .2 ft. = 6.5 ft.</u>

<u>Design HW is 6.5 ft. (outlet controls)</u> and the outlet velocity is 7.2 f.p.s. No heavy riprap is needed at the discharge apron.

<u>Inlet Control Problem</u>. The information necessary to solve this problem is given in Figure 8.4.3.

Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D = 1.08 from Chart 8.4.1.
<u>Then HW = 1.08 (5 ft.) = 5.4 ft.</u>

Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 132 ft. and type “C” inlet; H = 1.3 ft. from Chart 8.4.2. From Chart 8.4.3 critical depth = 3.4 ft. ho = (3.4 ft. + 5 ft.)/2 = 4.2 ft.
<u>Then HW = H+ho-Lso 4.2 ft. + 1.3 ft. - .7 ft. = 4.8 ft.</u>

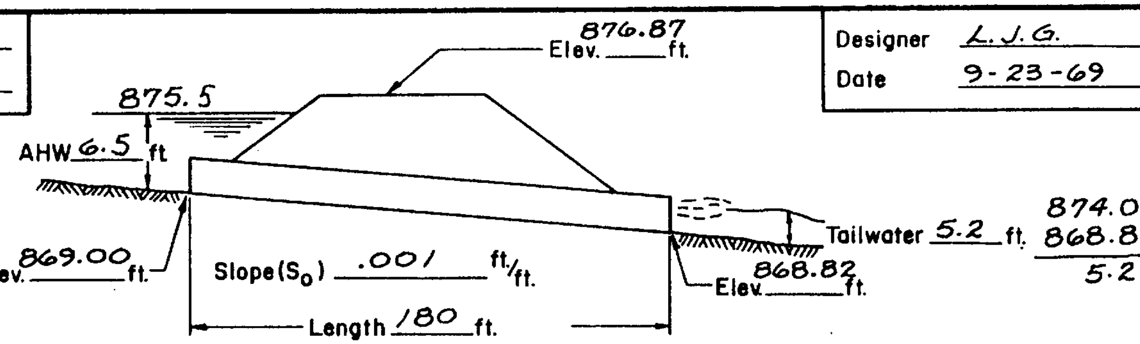
<u>Design HW = 5.4 ft. (inlet control)</u> and the outlet velocity from Chart 8.4.4 is 11.0 f.p.s. Heavy riprap is needed at the discharge apron.

HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
E-B-31-68

Project *Outlet Control Problem*
Culvert Sta. *560+00*

Designer *L.J.G.*
Date *9-23-69*



Hydrology:

(*50* freq.) Q = *720* cfs.

(____ freq.) Q = ______ cfs.

$\frac{L}{100 S_o} = \frac{180}{.1} =$ ______

Location comments: *The tailwater is controled by the inlet control problem which is downstream a short distance.*

Culvert			Q	Capacity Charts HW	Inlet Cont.		Outlet Control							Controling HW	Outlet Velocity f.p.s.	Comments
Entrance	Material	Size			$\frac{HW}{D}$	HW	K_e	d_c	$\frac{d_c+D}{2}$	h_o *	H	LS_o	** HW			
Type "C"	*R.C. Box*	*Twin 10x5*	*36 720*		*1.08*	*5.4*	*0.2*	*3.4*	*4.2*	*5.2*	*1.5*	*.2*	*6.5*	*6.5*	*7.2*	*(For n=.015)*

* h_o = The greater of $\frac{d_c+D}{2}$ or TW

** $HW = H + h_o - LS_o$

Summary & Recommendations:

FIGURE 8.4.2

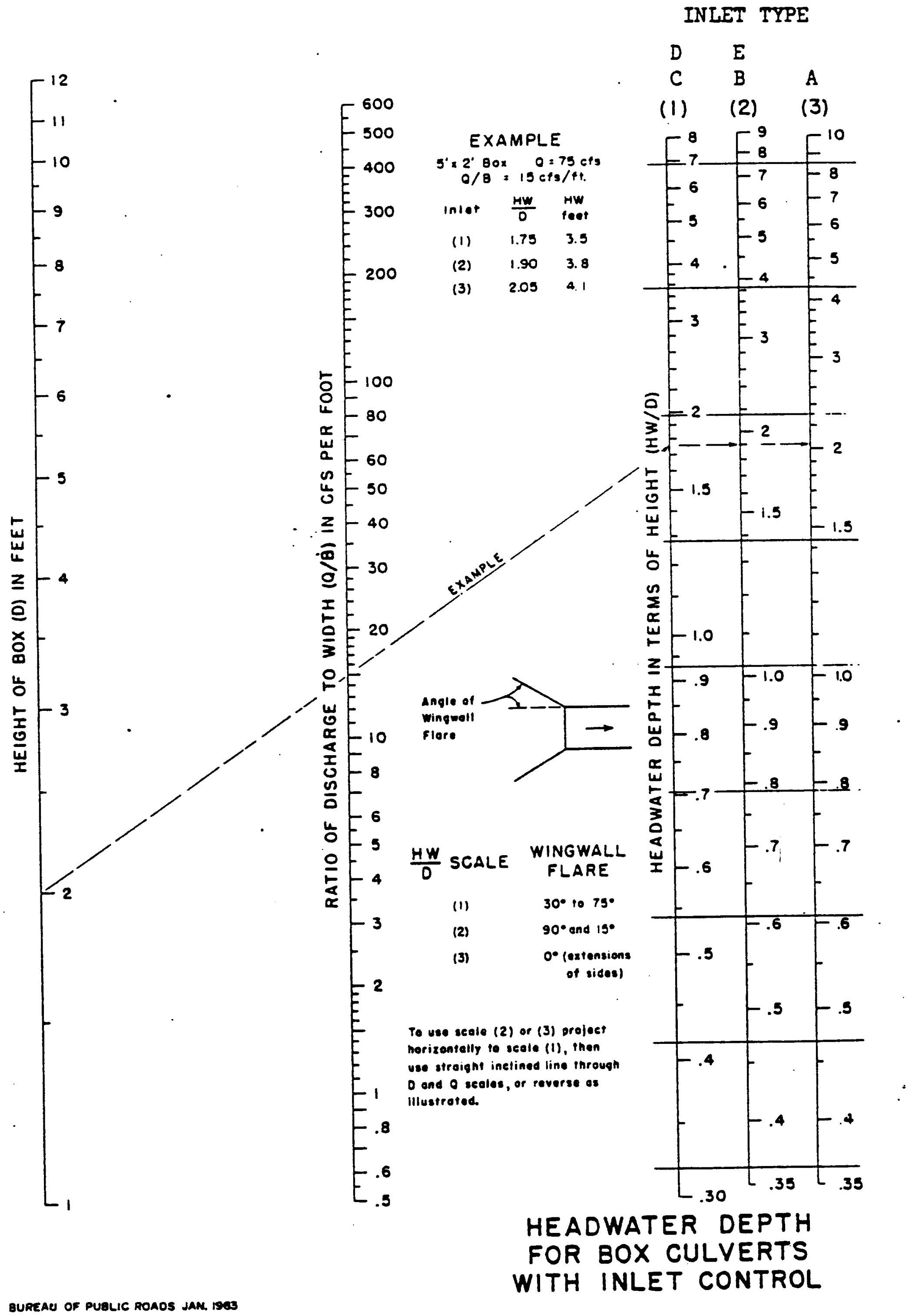
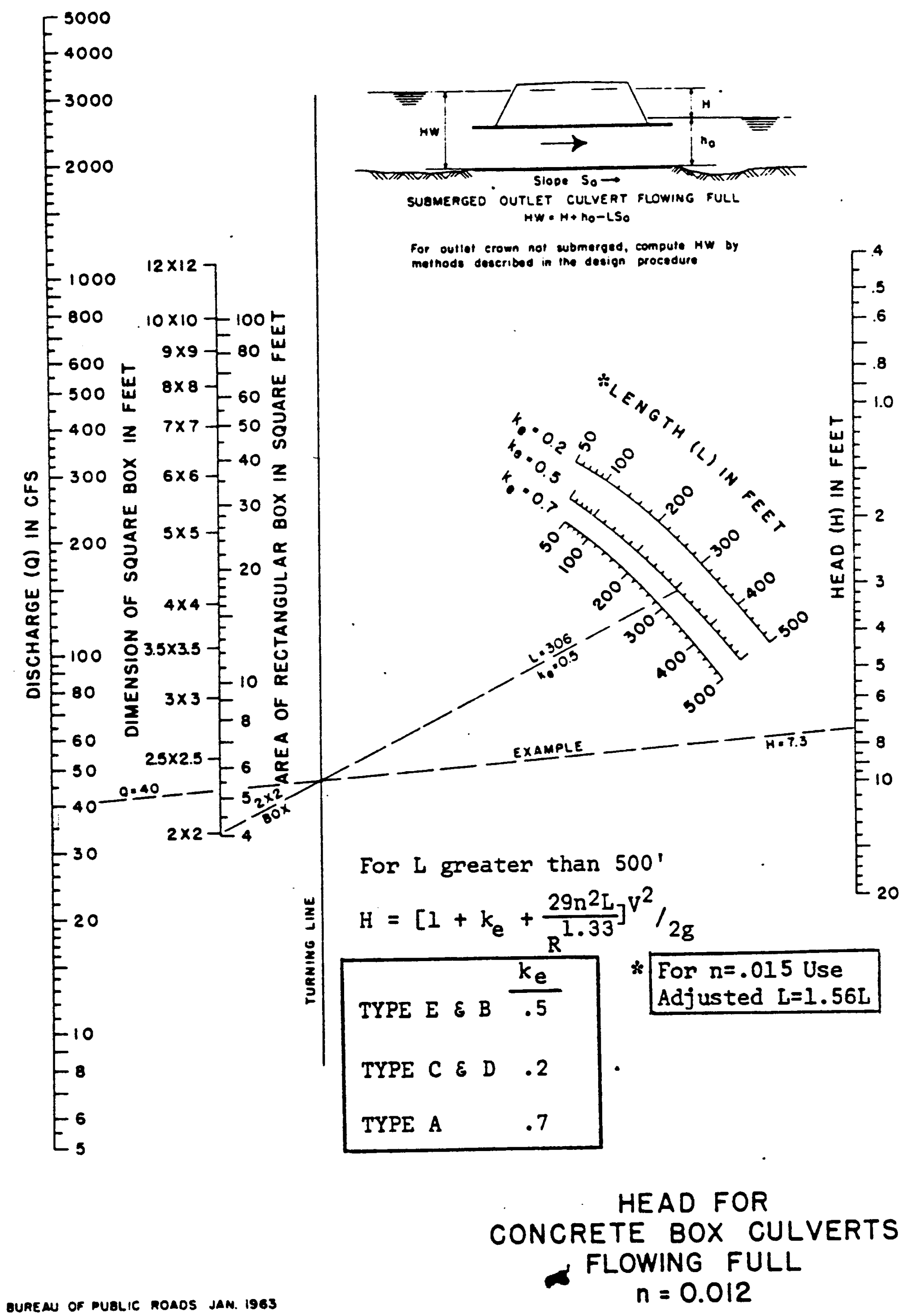

INLET TYPE
D E
C B A
(1) (2) (3)
EXAMPLE
5' x 2' Box Q = 75 cfs
Q/B = 15 cfs/ft.
Inlet HW/D HW feet
(1) 1.75 3.5
(2) 1.90 3.8
(3) 2.05 4.1
HEIGHT OF BOX (D) IN FEET
RATIO OF DISCHARGE TO WIDTH (Q/B) IN CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
EXAMPLE
Angle of Wingwall Flare
HW/D SCALE WINGWALL FLARE
(1) 30° to 75°
(2) 90° and 15°
(3) 0° (extensions of sides)
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL
BUREAU OF PUBLIC ROADS JAN. 1963

CHART 8.4.1



	k_e
TYPE E & B	.5
TYPE C & D	.2
TYPE A	.7

HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

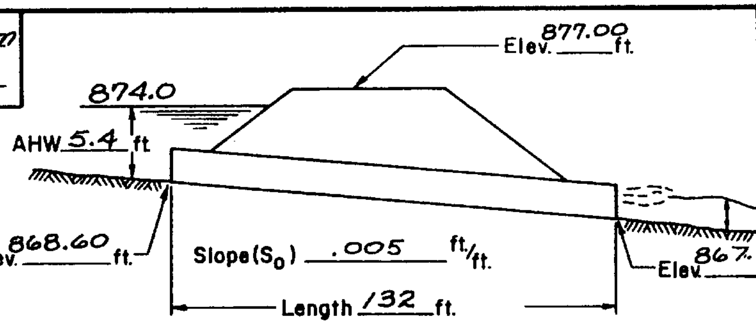
Chart 8.4.2

HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
E-B-31-68

Project *Inlet Control Problem*
Culvert Sta. *432+00*

Designer *L.J.G.*
Date *9-23-69*



Hydrology:

(*50* freq.) Q = *720* cfs.

(____ freq.) Q = ____ cfs.

$\frac{L}{100 S_o}$ = ________ = ________

Location comments: *This structure is located a short distance downstream of the outlet control problem.*

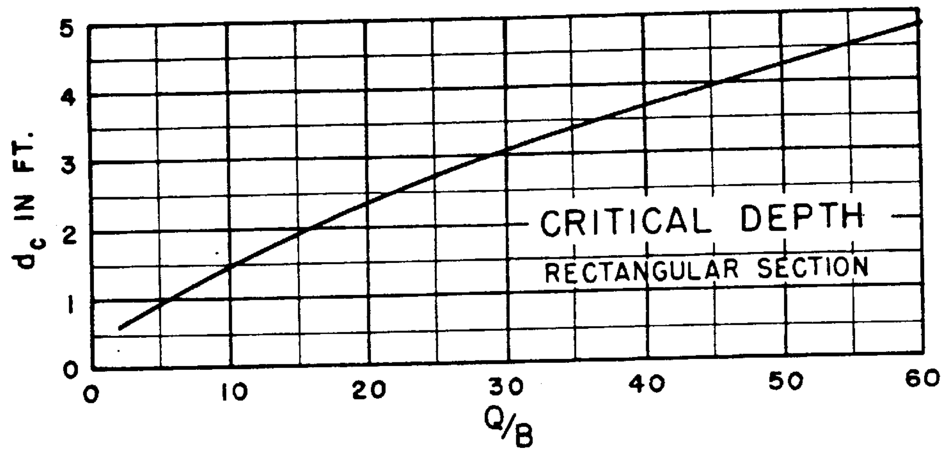
Culvert			Q	Capacity Charts HW	Inlet Cont.		Outlet Control							Controling HW	Outlet Velocity f.p.s.	Comments
Entrance	Material	Size			$\frac{HW}{D}$	HW	K_e	d_c	$\frac{d_c+D}{2}$	h_o*	H	LS_o	**HW			
Type "C"	R.C. Box	TWIN 10x5	36 720		1.08	5.4	0.2	3.43	4.2	4.2	1.3	.7	4.8	5.4	11.0	(n= .015)

* h_o = The greater of $\frac{d_c+D}{2}$ or TW

** $HW = H + h_o - LS_o$

Summary & Recommendations:

FIGURE 8.4.3


CRITICAL DEPTH
RECTANGULAR SECTION
d_c IN FT.
Q/B
0
1
2
3
4
5
0
10
20
30
40
50
60

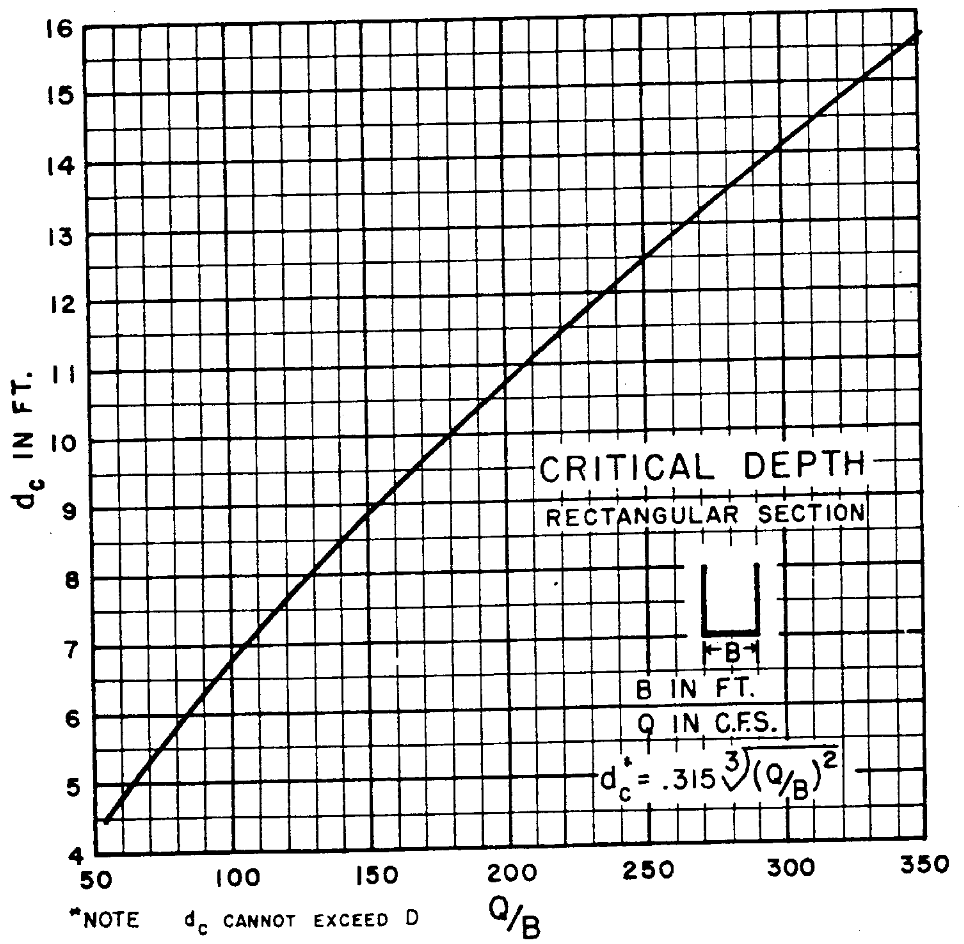

CRITICAL DEPTH
RECTANGULAR SECTION
B
B IN FT.
Q IN C.F.S.
$d_c^* = .315\sqrt[3]{(Q/B)^2}$
d_c IN FT.
Q/B
4
5
6
7
8
9
10
11
12
13
14
15
16
50
100
150
200
250
300
350
*NOTE d_c CANNOT EXCEED D
BUREAU OF PUBLIC ROADS JAN 1963

Chart 8.4.3

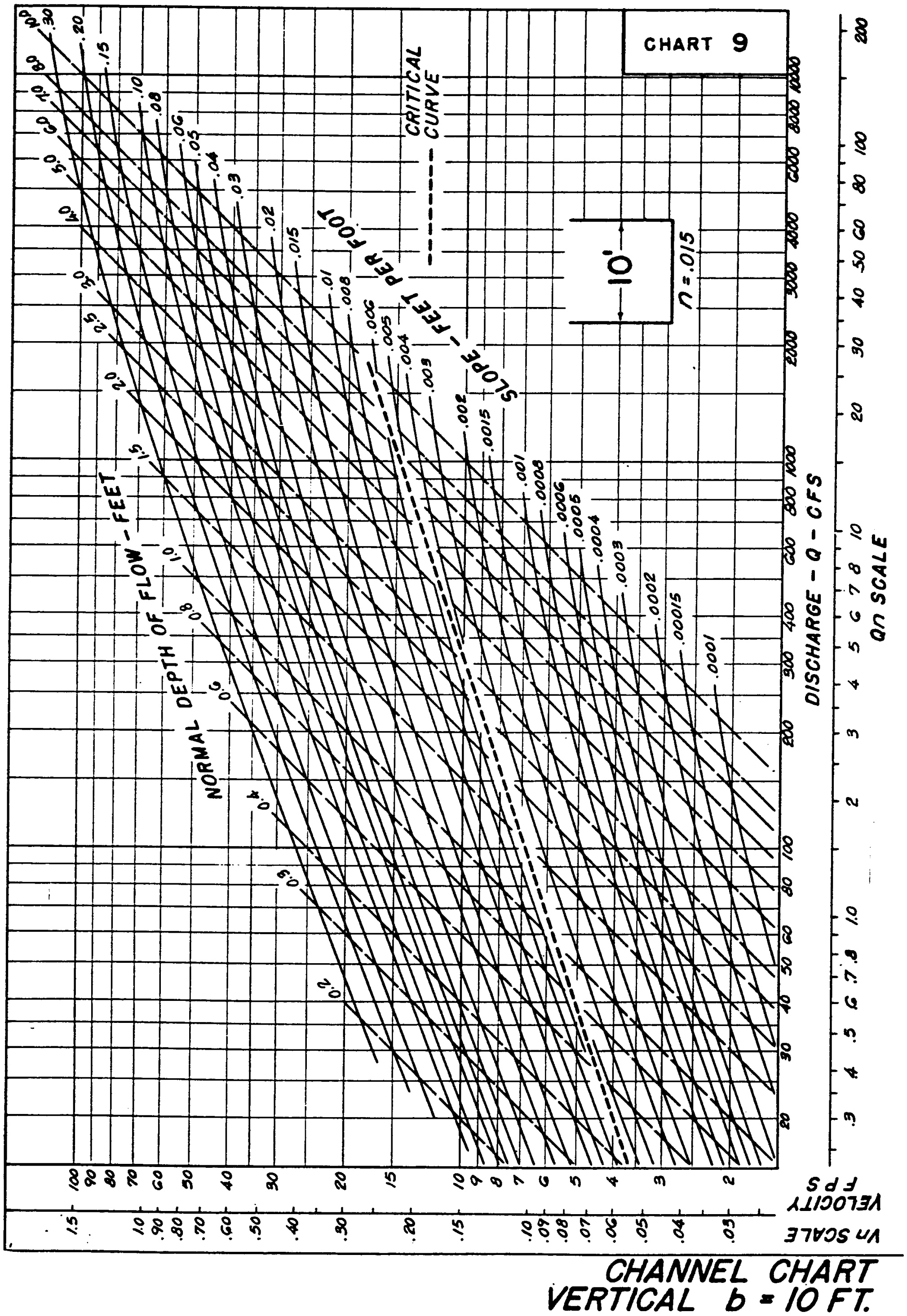


CHART 9
CRITICAL CURVE
SLOPE - FEET PER FOOT
NORMAL DEPTH OF FLOW - FEET
10'
n = .015
DISCHARGE - Q - CFS
Qn SCALE
VELOCITY F P S
Vn SCALE
CHANNEL CHART
VERTICAL b = 10 FT.

Chart 8.4.4

F. Roadway Overflow

See Section 8.3(2)F.

G. Outlet Scour and Energy Dissipators

Energy dissipating devices are used where it is desirable to reduce the discharge velocity by inducing high energy losses at the inlet or discharge ends of the structure. They are generally warranted when discharge velocities exceed 14 feet per second. Energy Dissipator designs shall be approved by the Bridge Engineer.

Energy losses may be induced at the culvert entrance with a drop inlet, or at the outlet using energy dissipating devices and stilling basins to form a hydraulic jump.

Drop inlets are used where headroom is limited, and energy dissipating devices and stilling basins at the discharge and are used where headroom is not critical.

For outlet devices utilizing the hydraulic jump, two conditions must be present for the formation of a hydraulic jump; the approach depth must be less than critical depth (supercritical flow); and the tailwater depth must be deeper than critical depth (subcritical flow) and of sufficient depth to control the location of the hydraulic depth. Where the tailwater depth is too low to cause a good hydraulic jump at the desired location, the required depth can be provided by either depressing the discharge apron or utilizing a broad-crested weir at the end of the apron to provide a pool of sufficient depth. The depressed apron method is preferred since there is less scouring action at the end of the apron. The amount of depression is determined as the difference between the natural tailwater depth and the depth required to form a jump.

There are numerous design concepts of energy dissipating devices and stilling basins that my be adapted for energy dissipation to reduce the velocity and avoid scour at the culvert outlet. The more common type of designs are drop inlets, drop outlets, chute spillways and riprap stilling basins.

More discussion on energy dissipators for culverts is available in references (3), (6), (7), (8). Four specific examples are shown below.

1) Drop Inlet. In drop inlet design, flow is controlled at the inlet crest by the weir effect of the drop opening. Drop inlet culverts operate most satisfactorily when the height of drop is sufficient to permit considerable submergence of the culvert entrance without submerging the weir or exceeding limiting headwater depths.

Referring to Figure 8.4.4, the general formula for flow into the horizontal drop opening is:

$$Q = C_1 (2g)^{1/2} L H^{3/2}$$

Where Q is the discharge in c.f.s., L is the crest length 2B+W, H is the depth of flow plus velocity head, and C_1 is a dimension discharge coefficient taken as 0.4275. The formula is expressed in english units as:

$$Q = 3.43 LH^{3/2}$$

and

$$L = Q/(3.43H^{3/2})$$

There are four connections which have to be multiplied times the discharge coefficient C, or times the factor 3.43:

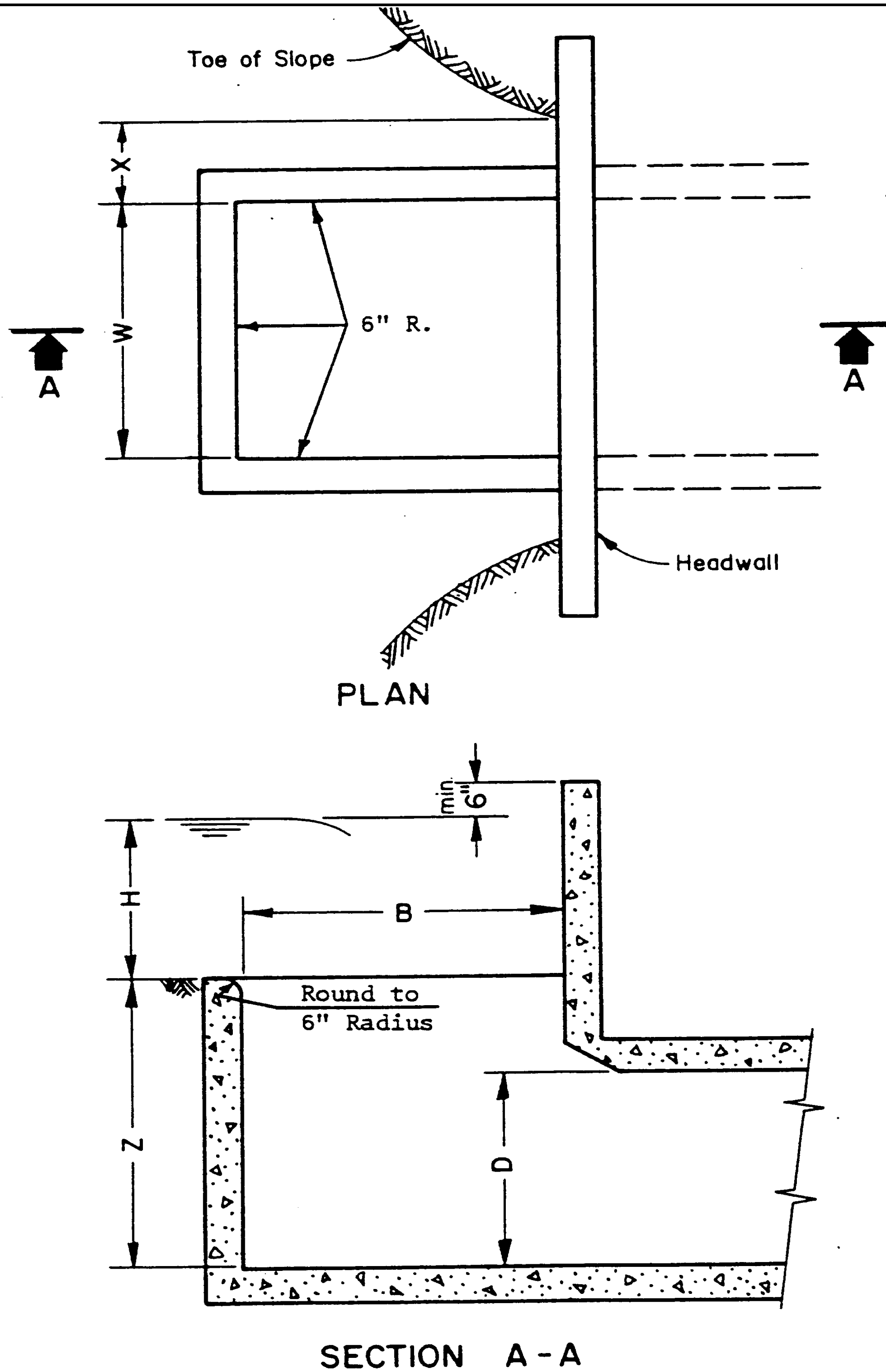
1. Correction for head H/W (Table 8.4.1)
2. Correction for box-inlet shape B/W. (Table 8.4.2)
3. Correction for approach channel width W_c/L (Table 8.4.3). (Wc = approach channel width = Area/Depth)
4. Correction for dike effect X/W (Table 8.4.4)

The size of the culvert should be determined by using the discharge (Q) and not allowing the height of water (HW) to exceed the inlet drop plus the critical depth of the weir which is given as:

$$d_c = [(Q/L)^2/g]^{1/3}$$

When using the hydraulic charts of Section 8.4(2)E, consider the culvert to have a wingwall flare of 0 degrees (extension of sides).

Sample computations are shown in Figure 8.4.5.



Box Inlet Drop Spillway
FIGURE 8.4.4

Table 8.4.1 Correction for head
(Control at box-inlet crest)
Multiply c_1 in Q = $c_1\, L\sqrt{2g}\, W\, {}^{3}\!/_{2}$ *by correction*

H/W	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0						0.76	0.80	0.82	0.84	0.86
.1	0.8	0.88	0.89	0.90	0.91	.91	.92	.92	.93	.93
.2	.93	.94	.94	.95	.95	.95	.95	.96	.96	.96
.3	.97	.97	.97	.97	.98	.98	.98	.98	.98	.98
.4	.99	.99	.99	.99	.99	.99	.99	.99	.99	1.00
.5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
.6	1.00									

Correction is 1.00 when H/W exceeds 0.6.

Table 8.4.2 Correction for box-inlet shape
(Control at box-inlet crest)
Multiply c_1 in $Q=c_1\, L\sqrt{2g}\, H^{3/2}$ by correction

B/W	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.98	1.01	1.03	1.03	1.04	1.04	1.03	1.02	1.01	1.01
1	1.00	.99	.99	.98	.98	.98	.97	.97	.96	.96
2	.96	.96	.95	.95	.95	.95	.95	.95	.94	.94
3	.94	.94	.94	.94	.94	.94	.94	.94	.93	.93
4	.93									

Table 8.4.3 Correction for approach-channel width
(Control at box-inlet crest)
Multiply c_1 in $Q=c_1 L\sqrt{2g} H^{3/2}$ by correction

W_c/L	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.09	0.18	0.27	0.35	0.44	0.53	0.62	0.71	0.80
1	.84	.87	.90	.92	.93	.94	.95	.96	.97	.97
2	.98	.98	.99	.99	.99	.99	1.00	1.00	1.00	1.00
3	1.00									

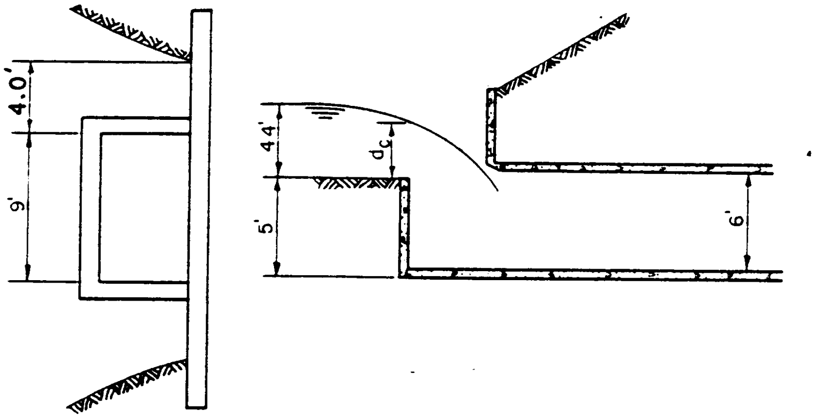
Correction is 1.00 when W_cL exceeds 3.0

Table 8.4.4 Correction for dike effect
(Control at box-inlet crest)
Multiply c_1 in $Q=c_1 L\sqrt{2g} H^{3/2}$ by correction

$\frac{B}{W}$	X/W						
	0.0	0.1	0.2	0.3	0.4	0.5	0.6
0.5	0.90	0.96	1.00	1.02	1.04	1.05	1.05
1.0	.80	.88	.93	.96	.98	1.00	1.01
1.5	.76	.83	.88	.92	.94	.96	.97
2.0	.76	.83	.88	.92	.94	.96	.97

DROP INLET
SAMPLE PROBLEM

Given: Q = 420 cfs through single 9'x6' box
H = 4.4' in a 27 ft. wide channel, drop = 5 ft.



Control at inlet crest: $L = \dfrac{Q}{3.43\ H^{3/2}}$ where Q = 420, H = 4.4

corrections:

1. $\dfrac{H}{W} = \dfrac{4.4}{9} = .49$; 1.00

2. (assume $B = \dfrac{W}{2} = 4.5$) $\dfrac{B}{W} = .5$; 1.04

3. $\dfrac{Wc}{L} = \dfrac{27}{9+2(4.5)} = \dfrac{27}{18} = 1.50$; .94

4. $\dfrac{X}{W} = \dfrac{4.0}{9.0} = 0.44$; 1.04

Total correction = 1.00x1.04x0.94x1.04 = 1.02

$$L = \frac{420}{1.02 \times 3.43 \times 4.4^{3/2}} = \frac{420}{1.02 \times 3.43 \times 9.23} = 13.01 < (2B+W=)\ 18\ ;\ \text{o.k.}$$

$$d_c = \sqrt[3]{\frac{Q^2}{L^2 g}} = \left(\frac{17.64 \times 10^4}{3.24 \times 3.22 \times 10^3}\right)^{1/3} = (16.85)^{1/3} = 2.56$$

HW must be less than $Z+d_c$ to prevent submerged wier.

With inlet control, $^{HW}/D = 1.19$ HW = 1.19x6 = 7.14

$7.14 < (5+2.56) = 7.56$, $\therefore$ weir controls

FIGURE 8.4.5

2) Drop Outlets

This generalized design is applicable to relative heights of fall ranging from 1.0 y/d to 15 h/d and to crest lengths greater than 1.5 d_c. Here y is he vertical distance between the crest and the stilling basin floor and d_c is the critical depth of flow.

$$D_c = 0.315[(Q/B)^2]^{1/3}$$

Referring to Figure 8.4.6 and Chart 8.4.5, this design uses the following formulas:

1. The minimum length L_b of the stilling basin is:

$$L_b = X_a + X_b + X_c = X_a + 2.55\ d_c$$

   a. The distance X_a from the headwall to the point where the surface of the upper nappe strikes the stilling basin floor is solved graphically in Chart 8.4.5.

   b. The distance X_b from the point at which the surface of the upper nappe strikes the stilling basin floor to the upstream face of the floor blocks is:

$$X_b = 0.8\ d_c$$

   c. The distance X_c, between the upstream face of the floor blocks and the end of the stilling basin is:

$$X_c \geq 1.75\ d_c$$

2. The floor blocks are porportioned as follows:

   a. The height of the floor blocks is:

$$0.8\ d_c$$

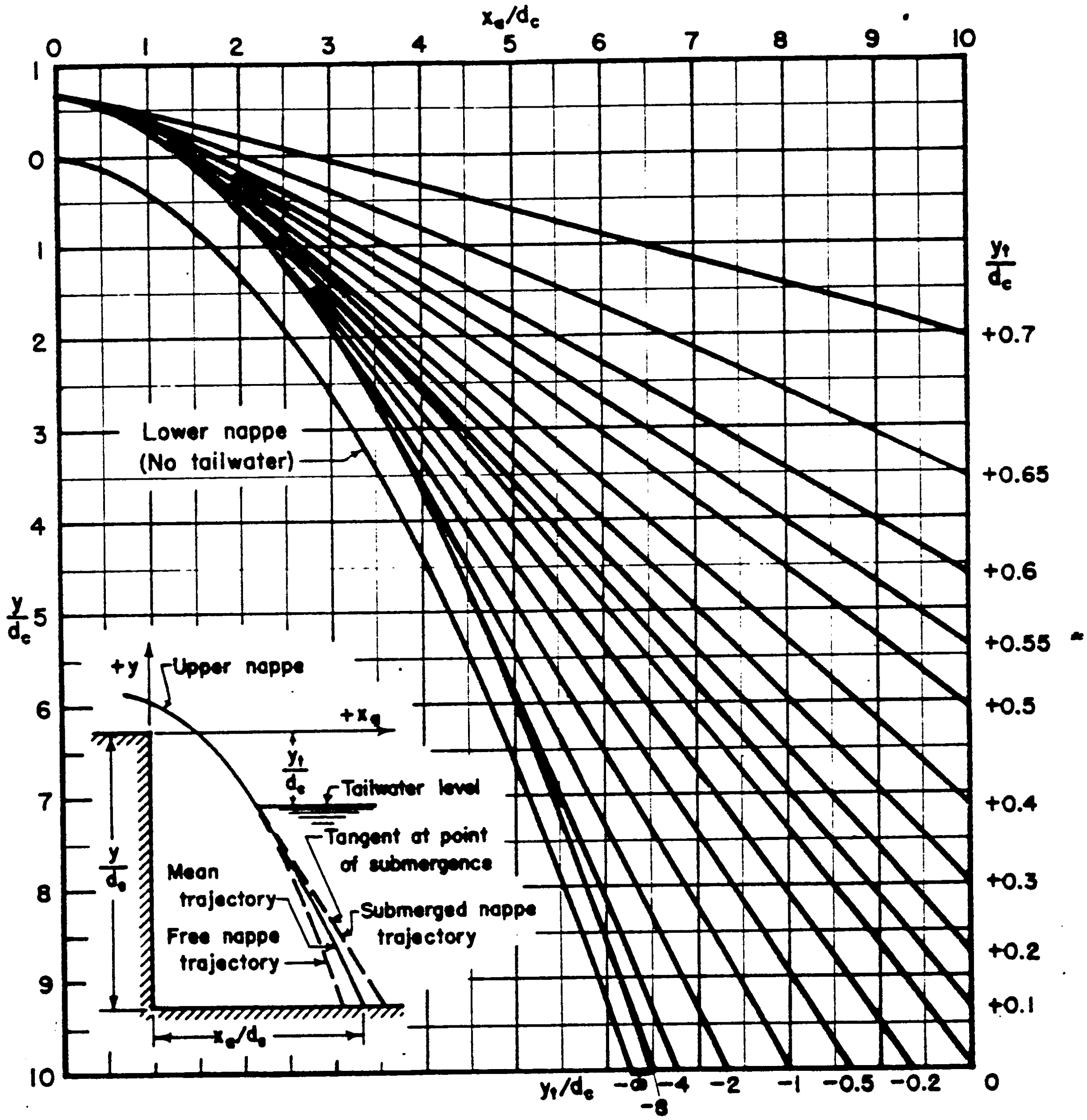
   b. The width and spacing of the floor blocks are approximately:

$$0.4\ d_c$$

   A variation of $\pm$ 0.15 d_c from this limit is permissible.

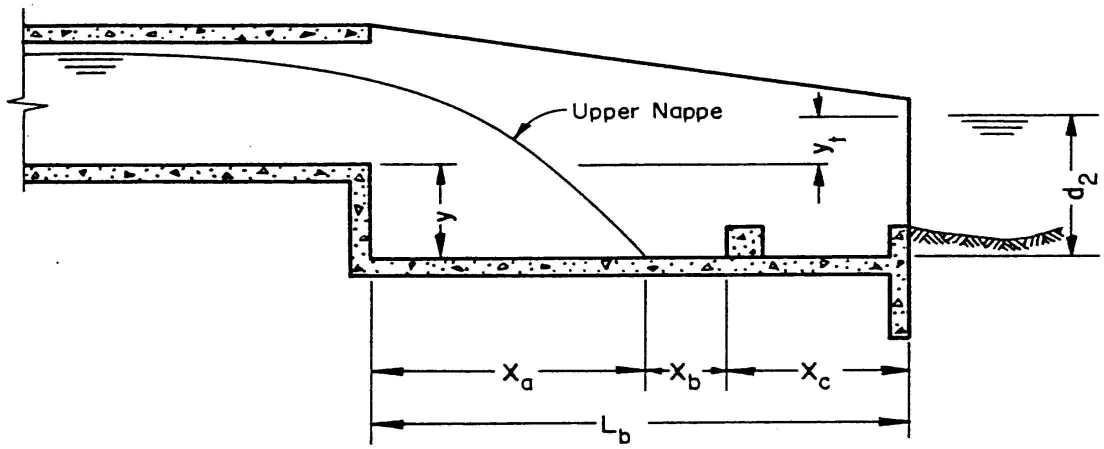
   c. The floor blocks are square in plan.

   d. The floor blocks occupy between 50 and 60 percent of the stilling basin width.

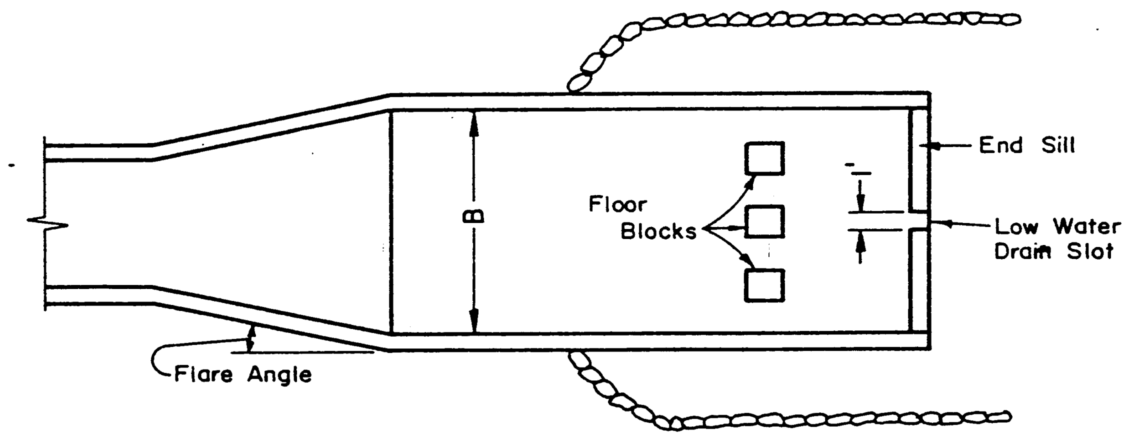


DESIGN CHART FOR DETERMINATION OF x_a

CHART 8.4.5



Section at Center Line



PLAN VIEW

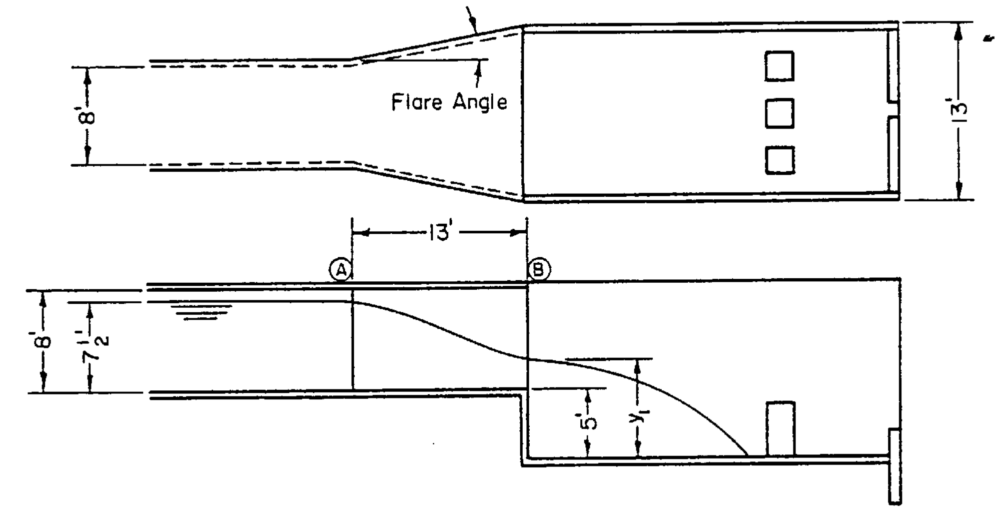
Straight Drop Outlet Spillway

FIGURE 8.4.6

DROP OUTLET

SAMPLE PROBLEM

Given: 8'x8' R.C. Box, 5' Drop, V in box = 13.5 fps, Depth in box = 7.5', Q = 800 cfs



Assume: That specific head at (A) is approximately equal to the specific head at (B). Therefore, elev. head + vel. head at (A) $\doteq$ elev. head + vel. head at (B)

End sill height should be less than or equal to 2'-0"

If the drop were placed at (A), $d_c = .315\sqrt[3]{(Q/B)^2} = .315(100)^{2/3} = 6.78$ and end sill = $0.4\, d_c \doteq 2'\text{-}9''$ which exceeds 2'. ∴ flare outlet.

To obtain 2' sill, $d_c = 2/.4 = 5' = .315(Q/B)^{2/3}$; $B = \left(\frac{.315 \times 800^{2/3}}{5}\right)^{3/2} = 13'$

flare from B = 8' to B = 13' at angle $^{150}/_{13.5} \doteq 11°$; length $= \left(\frac{13-8}{2}\right)/\tan 11° = 13'$.

Specific head $H_A = 7.5 + \frac{V_A^2}{2g} = 7.5 + \frac{13.5^2}{64.4} = 10.33'$

by trial & error; assume $\frac{V_B^2}{2g} = 7.5'$ $V_B = (64.4 \times 7.5)^{\frac{1}{2}} = 22$ fps.

elev. head (depth) = 10.33 − 7.5 = 2.83. Check trial: Q = AV = (13 × 2.83) × 22 = 809 ✓

Q = 800 ✓

$d_c = .315\sqrt[3]{(Q/B)^2} = .315\left(\frac{800}{13}\right)^{2/3} = .315 \times 15.6 = \underline{4.91'}$

$h_v/H = \frac{V_B^2/2g}{10.33} = \frac{7.5}{10.33} = .725$, which exceeds 1/3, ∴ $X_a^2 = \frac{2V^2}{g}y$,

$X_a = \left[\frac{2 \times 22^2 \times (5 + 2.83)}{32.2}\right]^{1/2} = 15.35$. Use $X_a = \underline{15'\text{-}6''}$

Dimensions:

Height of floor blocks	=	.8 x 4.91 = <u>4'-0"</u>
Height of end sill	=	.4 x 4.91 = <u>2'-0"</u>
Length of Basin	=	15.5 + 2.55 d_c = 28'
Floor blocks	=	<u>2'-0"</u> square
Height of sidewalls	=	(2.15 + .60) d_c = <u>13'-6"</u> above basin floor (use 13'-0")

FIGURE 8.4.7

3. The height of the end sill is:

$$0.4\ d_c$$

4. The sidewall height above the tailwater level is:

$$0.85\ d_c$$

5. The minimum height d_2, of the tailwater surface above the floor of the stilling basin is:

$$d_2 = 2.15\ d_c$$

In cases where the approach velocity head is greater than 1/3 of the specific head (velocity head + elevation head), X_a is checked by the formula and the greater X_a used.

$$X_a^2 = (2V^2/g)y_1$$

y_1 = top of water at crest
V = velocity of approach

Sometimes high values of d_c become unworkable, resulting in a need for large drops, high end sills and floor blocks. To prevent this d_c may be reduced by flaring the end of the barrel. The flare angle is approximately 150/V where V is the velocity at the beginning of the taper.

Sample computations are shown in Figure 8.4.7.

(3) Chute Spillways The simplest form of chute spillway has a straight centerline and is of uniform width. The outlet barrel of the culvert is sometimes flared to decrease y_1 so that the tailwater elevation necessary to cause a hydraulic jump need not be so high. This is done using the 150/V relationship as in the drop outlet sample problem. Y_1 is usually kept in the 2-3 foot range.

Referring to Figure 8.4.8, the required tailwater is computed by the formula:

$$y_2/y1 = ½\ [1+8F^2)^{1/2}) - 1]$$

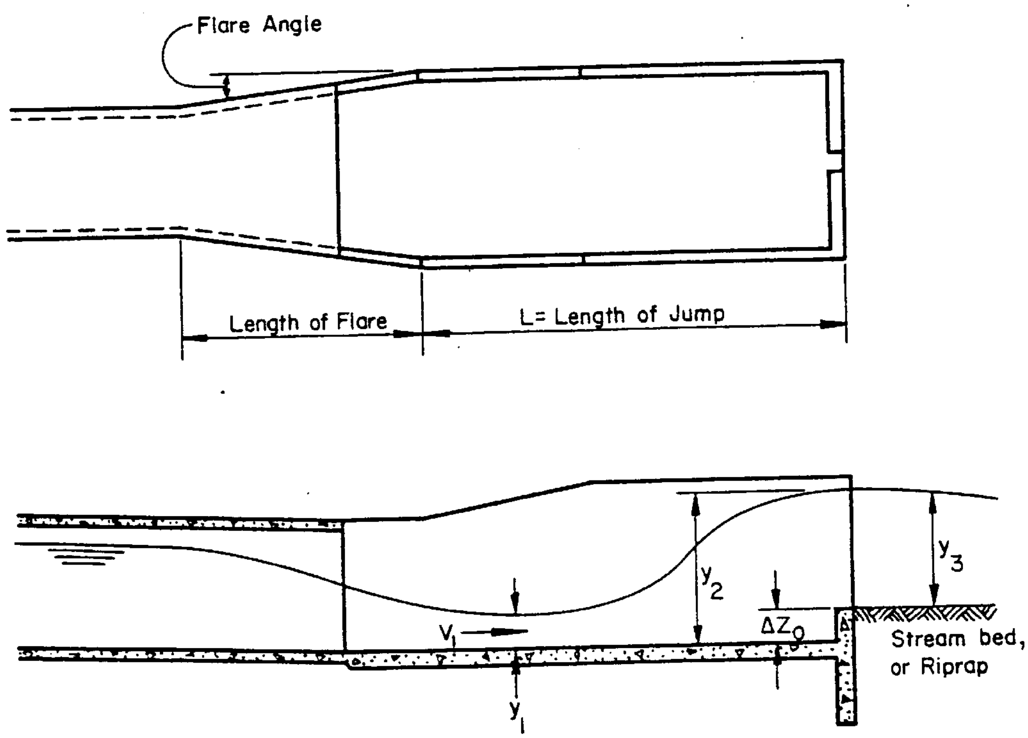


FIGURE 8.4.8
Chute Spillway

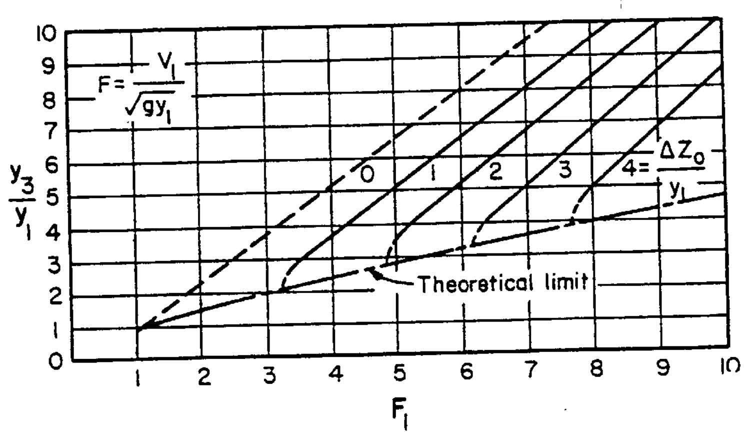


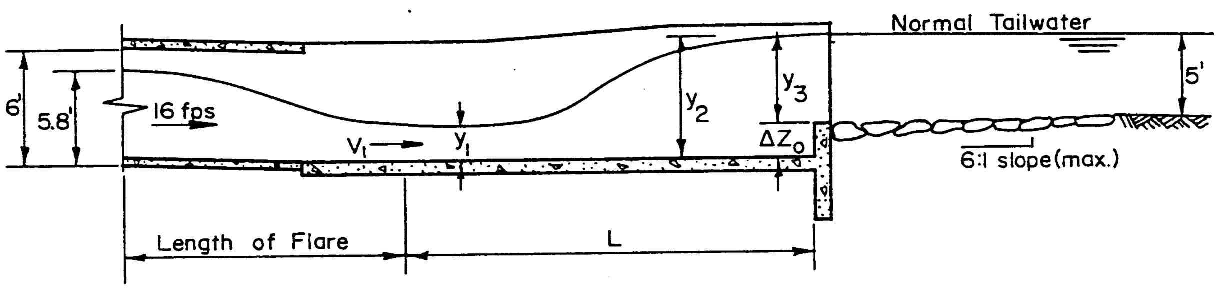
CHART 8.4.6

Characteristics of a Hydraulic Jump

at an Abrupt Rise

CHUTE SPILLWAY
SAMPLE PROBLEM

A discharge of 600 c.f.s. flows through a 7'x6' box culvert at 16 f.p.s. and a depth of 5.8'. Normal tailwater depth in the outlet channel is 5.0 feet.



Flare of wings = $\frac{150}{16} \approx 9°$

$$H = 5.8 + \frac{16^2}{2x32.2} = 5.8 + 3.975 = 9.775,\ Assume\ y_1 = 2.2^1, \frac{V_1^{\ 2}}{2g} = 7.575'$$

$$V_1 = (2x32.2x7.575)^{1/2} = 22.1\ fps$$

$$Q = 600 = AV = 2.2\ x\ width\ x\ 22.1,\ width = 12.36, \frac{12.36-7}{2} = 268$$

Length of flare = $\frac{2.68}{\tan 9°} = 17';\ y_1 = 2.20,\ V_1 = 22.1$

$$F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{22.1}{(37.2\,x\,2.2)^{1/2}} = 2.63$$

$y_2 = y_1$ x ½ $(\sqrt{1+8F_1^{\ 2}} - 1) = \frac{22.2}{2}(\sqrt{1+8\,x\,2.63^{-2}} - 1) = 7.15$

L = 6($y_2 - y_1$) = 6 (7.15 – 2.20) = 29.7' use L = <u>30</u> ft.

Assume y_3 = 5', y_3/y_1 = 5/2.2 = 2.27

From Chart 8.3.19, $\Delta Zo / y_1 = 0.5 \quad \Delta Zo = 1.1,\ use$ <u>1'-6"</u>

FIGURE 8.4.9

where y_2 = tailwater height required to cause the hydraulic jump,

F = Froude number = $v_1 /(gy_1)^{1/2}$

g = acceleration of gravity,

y_1 = velocity at beginning of jump.

End sill height (Z) is determined graphically from Chart 8.4.6.

Length of jump is assumed to be 6 times the depth change (y_2-y_1).

In many cases the tailwater height isn't deep enough to cause the hydraulic jump. To remedy this, the slope of the culvert may be increased to greater than the slope of the streambed. This will result in an apron depressed such that normal tailwater is of sufficient depth.

The problem of scour on the downstream side of the end sill can be overcome by providing riprap in the stream bottom. If riprap is used, it starts from the top of the sill at a maximum slope of 6:1 up from end sill to original streambed. If no riprip is used, the streambed begins at the top of the end sill.

Sample computations are shown in Figure 8.4.9.

4) Riprap Stilling Basins

The riprap stilling basins, in many cases, is a very economical approach to dissipate energy at culvert outlets and avoid damaging scour. A good treatise on riprap stilling basin is given in the FHWA *Hydraulic Design of Energy Dissipators for Culverts and Channels.* (6).

H. Select Culvert Design Alternatives

The “proposed culvert” design shall be based on several design factors. In most design situations, the pertinent hydraulic factors discussed in Section 8.4(1) will dictate the final selection of culvert size, length, scour protection, as well as the approach roadway design.

(3) References

1) U.S. Department of Transportation, Federal Highway Administration, *Hydraulic Design of Highway Culverts* Hydraulic Design Series (HDS) No. 5 Report No. FHWA-1P-85-15 September.

2) U.S. Department of Commerce, Bureau of Public Roads, *Debris-Control Structures*, Hydraulic Engineering Circular (HEC) No. 9 March, 1971.

3) U.S. Department of Interior, Bureau of Reclamation, *Design of Small Dam,* 1st Edition Washington D.C. 1961.

4) U.S. Department of Commerce, Bureau of Public Roads, *Design Charts for Open-Channel Flow* Hydraulic Design Series (HDS) No. 3, August, 1961.

5) U.S. Department of Transportation, Federal Highway Administration, *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 Report No. FHWA-IP-85-15 September, 1985.

6) U.S. Department of Transportation, Federal Highway Administration, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular (HEC) No. 14, September, 1983.

7) Blaisdell, Fred W. and Donnelly, Charles A., *Hydraulic Design of the Box Inlet Drop Spillway*, U.S. Department of Agriculture, Soil Conservation Service, SCS-TP-106, July, 1951.

8) Blaisdell, Fred W. and Donnelly, Charles A., *Straight Drop Spillway Stilling Basin*, University of Minnesota, St. Anthony Falls Hydraulic Laboratory, November, 1954.

APPENDIX 8-A **Sample Hydraulic/Site Report**

Site - Hydraulic Report
For
B-18-75
Short Street Bridge & Approaches
Chippewa River
City of Eau Claire
Eau Claire County
ID # 7995-01-77

Introduction

The city of Eau Claire and Wisconsin Department of Transportation propose to replace the existing narrow and structurally deteriorated bridge carrying Short Street over the Chippewa River in the City of Eau Claire. The project is located in Section 25, Town 27 North, Range 10 West.

This report describes and documents the Hydrologic and Hydraulic characteristics of the existing and proposed bridge. Also included in this report is a discussion of the design constraints and alternatives considered for the replacement of the subject bridge.

Data Available

Information in this report is based upon the following bulletins and publication:

1. "Flood-Frequency Characteristics of Wisconsin Streams" by Krug, Conger, & Gebert. US Geologic Survey Open File Report 91-4128.

2. "Drainage-Area for Wisconsin Streams" US Geologic Survey Open File Report 83-933

3. "Bridge Waterways Analysis Model: Research Report" Report No. FHWA/RD-86/108.

4. "Bridge Manual" by Wisconsin Department of Transportation.

5. Flood Insurance Study - City of Eau Claire, July 1984, FEMA

All river and floodplain Characteristics are based upon conditions observed November 13th, 1995. All elevations are referenced from National Geodetic Vertical Datum (NGVD) 1929.

Site Description

The site description of the proposed bridge replacement is illustrated in the following attachments to this report:

1. Stream Crossing Structure Survey Report
2. Preliminary Plan & Profile Sheet
3. 1" = 20' scale Contour Map
4. Project location maps
5. Preliminary structure plans
6. Original photographs
7. Geotechnical site report
8. Flood Insurance Study exhibits

The size of the Chippewa River Drainage basin at the proposed site is approximately 6620 sq. mi. Both upstream and down stream reaches of the river are well defined with all normal flows being confined within a definite Floodway. The adjacent upstream and downstream river banks are lined with trees, steep banks and private residences. There are sandstone outcroppings in the surrounding vicinity and the stream bed material consists of sand, gravel and boulders. The normal flow channel width is generally uniform in the vicinity of the project and measures approximately 460 feet.

The Chippewa River at the project site is designated as a Flood Hazard Area (FHA) on the City of Eau Claire Flood Insurance Rate Map (FIRM) dated July 17, 1984. This FHA is a Detailed Study area with an associated Regulatory Profile.

Based upon field measurements, the water surface slope of the channel is approximately 0.011%. This slope appears consistent with both the FIS flood profiles and slope computed from the USGS Quadrangle.

The upstream bridge , B-18-114 located 0.37 mi. upstream carries Clairemont Ave. over the Chippewa River. This 70 inch prestressed girder structure built in 1993 has seven spans and an overall length of 846 feet. As built bridge plans indicate that the hammer head piers are supported on end bearing piles 30 to 46 feet long.

The downstream Bridges , B-18-23&24 located 0.75 mi. downstream carries Interstate Highway 94 over the Chippewa River. These bridges are 7 span plate girder structures built in 1964 and have overall lengths of 915 feet. As built plans indicate that the substructure is supported on end bearing piles 36 to 69 feet long.

Magnitude and Frequency of Floods

In the vicinity of the project site, the Chippewa River is a highly regulated and gaged river. A flood frequency vs. discharge analysis was made to estimate the design discharge for the 100 year flood **(Design Event)** . The analysis methods consisted of examination of flood estimates base on USGS 1992 Regression Equations, Three gages on the Chippewa River (Log-Pearson III 100-year flood results) transferred to the project site, and the City of Eau Claire 1984 Flood Insurance Study. Regional -regression equations are not good estimators on regulated streams, therefore, little weight was given to their estimates. The three gages on the Chippewa river had drainage areas that ranged from 85% to 136% of the project sites drainage area. The transferred flows from these gages yielded a consistent estimate of about 99,700 cfs. The city of Eau Claire FIS indicated a Regulatory 100-year flow value of 131,000 cfs. A 100-year Design Discharge of 99,700 cfs was elected because of the extensive gage information and consistency. However, Regulatory Flood impacts will be examined and reported base on the adopted City of Eau Claire FIS 100-year flood of 131,000 cfs.

Hydrologic calculation were previously submitted on August 12, 1995. The Design Discharge of 99,700 cfs was approved by WisDOT Central Office Bridge on September 9, 1995.

Analysis of Hydraulic Characteristics

The hydraulic characteristics of the floodplain for both existing and proposed conditions were calculated in metric units using the FHWA - WSPRO step backwater program. Cross-section were developed from contour mapping and surveyed information. Manning roughness coefficients were determined from field observations and photographs. The existing FIS - HEC-2 model was examined for use in hydraulic calculation. However, do to questionable bridge definition at the project site it was decided to use an independent WSPRO model that incorporated the most recent survey information. The tailwater elevation for both the design flow of 99,700 cfs and the Regulatory Flow of 131,000 cfs is based on the FIS profile rating curve downstream of the project. It has been decided not to elect the conservative FIS Regulatory 100-year flow for design purposes because this conservative flow would preclude the use of economical replacement alternatives based on minimum freeboard requirements. Therefore, both Design and Regulatory existing and proposed 100-year flood profiles have been calculated.

Existing Structure and Approaches

The existing 3-span overhead truss built in 1924 has spans of 175-177-175 feet between supports. The abutments are vertical face full retaining. The piers are tapered solid shafts. The west abutment and pier 1 are on spread footing while pier 2 and the east abutment are supported on timber end bearing piles. The roadway width between curbs is 18.8 feet and both the superstructure and substructure are in poor condition. There is one sidewalk on the existing structure on the south side that is 6.5 feet clear. There have been no reports of road overflow or ice/debris problems at the site. Examination of the contour mapping indicates that there may be existing local scour around the piers.

At the 100-year Design flood discharge of 99,700 cfs, all of the flow passes through the bridge opening in low flow with no road overflow. The design flow utilizes an area of 12,885 sq. ft. and has an average velocity of 7.74 fps. The existing 100-year design high-water is 773.85 feet with an associated backwater of 0.03 feet.

Discussion of Structure Sizing

The potential for damage to many adjacent private properties due to flooding on the Chippewa River at this site is quite high. The replacement structure should not cause additional backwater because of the extensive risk to the adjacent properties. Because of the observed local scour holes in the vicinity of the piers, the proposed structure should be designed to resist and withstand the potential of scour.

The propose tangent horizontal alignment across the bridge will be maintained while the proposed vertical alignment will be raised to facilitate the use of deeper super structure. The proposed roadway will be 32 feet clear and there will be on sidewalk 10.0 feet clear on the south side of the structure. The City of Eau Claire has requested that lighting be placed on the south side of the bridge along the sidewalk.

Due to the shallow bedrock on the east side of the river, substructure units will have to be founded on spread footings while units on the west side of the river will be founded on piles. Construction of coffer dams in the river may be difficult and expensive due to the shallow bedrock, therefore the number of piers should be minimized.

The WisDNR is satisfied with the existing navigational clearance and has indicated that no threatened or endangered resources are located in the project vicinity. WisDNR has requested that fill into the river be minimized and that proper erosion control be exercised during construction.

Alternates Considered

Rehabilitation of the existing structure is not feasible do to the type of structure and its poor condition.

Reinforced concrete flat and haunched slabs were eliminated as uneconomical alternatives due to the number of spans and substructure units that would be required.

A 4-span 70 inch prestressed girder or 4-span steel plate girder structure were determined to be the most feasible and economic alternatives for this site. Both of these alternative would have the same span geometry (124 - 151 - 151 - 124 feet and would utilize the same type and location of substructure units. Both structures would also have the same roadway cross section. Also, both alternatives exceed the 2.0 feet of freeboard over the Design 100-year flood elevation. The 70 inch prestressed girder alternatives interior spans of 151 feet will require 8000 psi concrete based on initial transfer and final stresses. Use of the higher strength concrete is justified by the cost savings associated with longer spans and fewer piers. The use of the higher strength concrete was previously discussed and approved by the Bridge Office.

Estimated cost of the prestressed and steel girder alternatives was based on similar let structure cost and statewide average unit bridge cost shown in chapter 5 of the bridge manual. The cost of the 4-

span steel plate girder alternative was estimated to be $1,352,000 while the cost of the 4-span 70 inch prestressed girder alternative was estimated to be $1,112,000. The prestressed concrete girder alternative is estimated to cost $240,000 less than the steel plate girder alternative based on initial construction. The concrete prestressed girder alternative is also expected to require less maintenance cost during the life of the structure.

Based on the site requirements, initial cost and long-term maintenance cost, a 4-span 70 inch concrete prestressed girder structure is the proposed alternative for this crossing.

Proposed Structure and Approaches

The proposed structure is a 4-span 70 inch prestressed girder structure supported on A-3 abutments and hammer-head piers. The span geometry will be 124-151-151-124 feet between center of supports. The structure will not be skewed. The roadway will be 32 feet clear with one 10.0 foot sidewalk located on the south side separated from traffic with a modified parapet ‘LF’. There will be a 5 foot steel rail along the exterior of the sidewalk and light standards located above each pier. The structure will support gas, telephone, and water utilities between the girders.

The structure will be located on the existing horizontal tangent alignment. The vertical profile will be raised to provide better approach geometrics and to provide adequate freeboard over the design 100-year flood elevation.

For the 100-year design event of 99,700 cfs, the structure provides 12,842 sq. ft. of waterway area with an associated velocity of 7.74 fps. The proposed design 100-year high-water of 773.85 feet matches the existing 100-year high water of 774.85 feet. The proposed structure provides 5.8 feet of freeboard over the design 100-year high-water elevation. The Bridge has been evaluated for scour and assigned a scour code of 5 (stable for calculated scour).

Summary of Hydraulic Characteristics

The hydraulic data for the existing and proposed bridge is summarized as follows:

	Existing Bridge	Proposed Bridge
Drainage Area (sq. mi.)	6620	6620
Design 100-year Flow (cfs)	99,700	99,700
Design 100-year High-water (ft.)	773.85	773.85
Waterway Area (sq. ft.)	12,885	12,842
Velocity Through Bridge (mps)	7.74	7.74
Overflow Frequency (yr.)	> 100-year	>100-year

Regulatory Profile (City of Eau Claire 1984 FIS)

Regulatory 100-year Flow (cfs)	131,000	131,000
Regulatory Tailwater Elevation (ft.)	775.79	775.79
Regulatory 100-year High-water (ft.)	776.54	776.54

February 11, 1988

APPENDIX 8-B

TO: Department of Transportation Division Administrators
Department of Natural Resources District Directors

FROM: Ronald R. Fiedler, P. E.
Secretary
Department of Transportation

Carroll D. Besadny
Secretary
Department of Natural Resources

SUBJECT: Implementation of DOT/DNR Cooperative Agreement, Section VII
Waterway Crossings and Other Floodplain Encroachments

In 1983, DOT and DNR Secretaries signed a revision to the DOT/DNR Cooperative Agreement covering DOT projects that involve waterway crossings and other floodplain encroachments. The agreement was never fully implemented because the procedures for landowner notification were not spelled out in the agreement. The purpose of this memo is to specify the procedures agreed to implement all elements of the agreement relating to waterway crossings and other kinds of floodplain encroachments.

Effective immediately, the provisions of the Cooperative Agreement relating to waterway crossings and other floodplains encroachments will be implemented by both agencies with the following clarification:

1. The agreement applies to ALL DOT development projects which cause an encroachment into mapped flood hazard areas. Included are ALL culverts, bridges, and fills within any area mapped as floodplain.

2. If the staff of either agency believes a DOT project within a mapped floodplain area, for reasons of size or significance, should not be reviewed under the agreement, OR if either agency believes a project in an unmapped area should be reviewed under the agreement, they should discuss their concern with the other agency. Upon agreement of both district or both agency liaisons, any DOT project can be excluded from, or included in the requirements of this agreement. All such agreements shall be documented at some point before completion of the liaison process.

3. In mapped areas, DOT will, unless agreed above, compute the 100-year regional flood discharge and elevation on all new or replacement waterway crossing structures and for all other floodplain encroachments, including culverts, in accordance with NR 116 and NR 320. DOT will provide the results of the computations to the identified DNR district liaison. In all cases (no flood level increase, reduced flood levels, or increased flood levels), DOT will ensure the appropriate zoning administrator is notified.

   In unmapped areas, the results of any flow or backwater calculations that are computed by DOT for design or other purposes will be provided to the DNR District Environmental Impact Coordinator as a cooperative courtesy. Other than agreed upon under (2) above, the unmapped area projects will not be reviewed under this specific agreement.

4. If an increase in backwater will result (mapped areas only or where otherwise agreed), the DOT district will notify ALL affected landowners upstream from the project by certified letter, return receipt requested. The notification will be made using one of the three attached form letters. Letter 1 will be sent to affected property owners where no floodplain ordinance is in effect. Letter 2 will be sent to affected property owners where a floodplain zoning ordinance is in effect. Letter 3 will be sent to affected property owners whose property DOT has determined will be diminished in value by the increased flood water elevation resulting from the DOT project.

   The determination whether a floodplain zoning ordinance is in effect will be made by DNR early in the liaison process and will be transmitted to DOT following notification to DNR of the project. In most cases, the "Floodplain Management Community Status Report," issued twice annually by DNR will be used to make this determination.

   DOT will send the DNR Environmental Impact Coordinator reviewing the project a copy of ALL landowner notification letters. Copies of the certified mail receipt and its return receipt should be retained with the district's project files. The form letters are not to be amended except to include appropriate identification information. A copy of Section 88.87, Wis. Stats. will also be sent to affected landowners as part of the notification. Any other information DOT may wish to convey to a landowner will be handled separately.

5. Where a floodplain ordinance is in effect, "appropriate legal arrangements" will be required. In addition to the notification letter, one of the following must be done prior to project construction to comply with this requirement:

   a. Acquisition of property rights (fee title or easement), or provision of other compensation agreed to in writing by the property owner.

b. Initiation of condemnation proceedings as provided in Chapter 32, Wis. Stats.

c. Receipt of a document signed by the property owner (response sheet to Letter 2), or a certified letter return receipt, which verifies the property owner has received the applicable notification letter (Letter 2). To keep the DNR Environmental Impact Coordinator informed of comments received from these specific property owners, a copy of all completed or partially completed returned response sheets must be sent to the DNR coordinator.

It is agreed that DOT is the agency responsible for determining which of the requirements (a, b, or c) is appropriate for a project.

6. As indicated in Section VII of the DOT/DNR Cooperative Agreement:

DOT project development scheduling normally provides sufficient lead time for the zoning ordinance amendment process to be completed prior to construction.

Upon notifying DNR, the local unit of government, and the appropriate floodplain zoning authority of the predicted increase in the height of the regional flood, and making appropriate legal arrangements with affected property owners, and providing evidence of this to DNR, DOT or its authorized agent may proceed with project development.

DNR shall provide timely assistance to local units of government in the development, adoption, and administration of their official floodway lines, water surface profiles, floodplain zoning maps, and zoning ordinances consistent with their authority and responsibility under NR 116.

DNR shall notify DOT in a timely manner about any significant problems which might arise during the ordinance amendment process that might indicate reconsideration of the project development schedule. If such a situation arises, DOT and DNR shall resolve these jointly on an individual basis pursuant to the spirit and intent of this Agreement.

If a community fails to amend its ordinance in a timely manner (six months after the new regional flood elevation is made available to local officials and affected landowners) or denies the amendment, even though the new floodplain information has been provided and appropriate legal arrangements have been made with affected property owners, DOT may proceed with project construction after consultation with DNR.

LETTER 1
LETTER FOR UNZONED FLOODPLAINS

[Property Owner]
[Inside Address]

[Highway Project ID Information]

The (WisDOT or local government) is planning a highway (highway number, letter or other designation) bridge (if planned structure is not a bridge appropriately identify the structure) over (identify waterway). The new bridge (or other planned structure) may at some time increase the flood water elevation on your property.

Flood water elevations are estimated using the predicted 100-year flood elevation. The predicted 100-year flood elevation is the best estimate of the highest flood water elevation that will likely occur during a 100-year period; it is a flood elevation that has a 1 in 100 chance of being reached in any given year. The predicted 100-year flood elevation for your property following construction of the planned bridge (or other structure) will be (inches or feet) higher than the current predicted 100-year flood elevation for your property.

Our review shows that this increase in the predicted 100-year flood water elevation for your property does not diminish your property's value or usefulness and will not result in any damage to you for which the law entitles you to be paid. You are, however, advised of section 88.87(1) and (2), Wisconsin Statutes, a copy of which is included for your information. In relevant part, that statute permits a property owner to file a claim for damage resulting from unreasonable or unnecessary water accumulation from highway construction that unreasonably impedes water flow. The damaged property owner must file a claim within 90 days after the alleged damage occurs.

Sincerely,

(DOT or Authorized Agent Signature)

Enclosure

cc: DNR District Office
Local Government

LETTER 2
LETTER FOR ZONED FLOODPLAINS

[Property Owner]
[Inside Address]

[Highway Project ID Information]

The (WisDOT or local government) is planning a highway (highway number, letter or other designation) bridge (if planned structure is not a bridge appropriately identify the structure) over (identify waterway). The new bridge (or other planned structure) may at some time increase the flood water elevation on your property.

Flood water elevations are estimated using the predicted 100-year flood elevation. The predicted 100-year flood elevation is the best estimate of the highest flood water elevation that will likely occur during a 100-year period; it is a flood elevation that has a 1 in 100 chance of being reached in any given year. The predicted 100-year flood elevation for your property following construction of the planned bridge (or other structure) will be (inches or feet) higher than the current predicted 100-year flood elevation for your property.

Our review shows that this increase in the predicted 100-year flood water elevation for your property does not diminish your property's value or usefulness and will not result in any damage to you for which the law entitles you to be paid.

A prepaid, self-addressed envelop is enclosed for your convenience should you choose to respond on the enclosed form concerning this matter. Any response should be made within 15 days from the date of this letter. Your failure to respond will indicate you have no comment but will not prevent you from pursuing any lawful claim you may have in the future.

You are advised of section 88.87(1) and (2), Wisconsin Statutes, a copy of which is included for your information. In relevant part, the statute permits a property owner to file a claim for damage resulting from unreasonable or unnecessary water accumulation from highway construction that unreasonably impedes water flow. The damaged property owner must file a claim within 90 days after the alleged damage occurs.

Wisconsin Administrative Code, Chapter NR 116, requires that any time an action occurs that causes an increase of more than one-hundreth of a foot in the 100-year flood elevation, a zoning change is required. This change is necessary to assure that local zoning administrators place on record the latest available information on the 100-year flood elevation. Appropriate action will soon be taken to record the new 100-year flood elevation for the floodplain in which your property is located.

Sincerely,

(DOT or Authorized Agent Signature)

Enclosure

cc: DNR District Office
Local Government

FORM FOR ENCLOSURE WITH LETTER 2

Response

[ ] I wish the following additional information:

[ ] I have the following questions, comments, or concerns:

[ ] I have no further comments, questions, or concerns.

(Date) (Signature)

NOTE: It is recognized that the property owner may or may not choose to sign.

LETTER 3
DRAFT LETTER FOR
CIRCUMSTANCES WHERE COMPENSATION IS REQUIRED

[Property Owner]
[Inside Address]

[Highway Project ID Information]

The (WisDOT or local government) is planning a highway (highway number, letter or other designation) bridge (if planned structure is not a bridge appropriately identify the structure) over (identify waterway). The new bridge (or other planned structure) may at some time increase the flood water elevation on your property.

Flood water elevations are estimated using the predicted 100-year flood elevation. The predicted 100-year flood elevation is the best estimate of the highest flood water elevation that will likely occur during a 100-year period; it is a flood elevation that has a 1 in 100 chance of being reached in any given year. The predicted 100-year flood elevation for your property following construction of the planned bridge (or other structure) will be (inches or feet) higher than the current predicted 100-year flood elevation for your property.

Our review shows that this increase in the predicted 100-year flood water elevation for your property diminishes your property's value. The predicted increase in flood water elevation for your property will result in damage to your property when the bridge (or other structure) is constructed. You are, therefore, entitled to appropriate compensation. Hence, you have been or will be contacted about the property interest that should be acquired from you and about the compensation that should be paid to you as a consequence of the predicted 100-year flood water elevation increase for your property.

Finally, you are advised of section 88.87(1) and (2), Wisconsin Statutes, a copy of which is included for your information. In relevant part, the statute permits a property owner to file a claim for damage resulting from unreasonable or unnecessary water accumulation from highway construction that unreasonably impedes water flow if compensation for the damage has not already been paid. The damaged property owner must file a claim within 90 days after the alleged damage occurs.

Sincerely,

(DOT or Authorized Agent Signature)

Enclosure

cc: DNR District Office
Local Government

COOPERATIVE AGREEMENT
BETWEEN
WISCONSIN DEPARTMENT OF TRANSPORTATION
AND
DEPARTMENT OF NATURAL RESOURCES
(amended July 1995)

I. Statement of Purposes

The Wisconsin Department of Transportation (DOT) and the Department of Natural Resources (DNR) recognize that the Wisconsin Legislature has charged DNR with the responsibility for protecting the State's land, water, fish and wildlife resources; and has charged DOT with furnishing the citizens of Wisconsin with an adequate, safe and economical transportation system. The DOT and DNR further recognize that the construction, reconstruction, maintenance and repair of transportation facilities, including highways and bridges, may have potentially adverse effects on the environment.

Therefore, the DOT and DNR agree that in the interest of fulfilling their respective duties, and to provide a reasonable and economical procedure for carrying them out in a manner that is in the total public interest, the DOT and DNR will consult and cooperate with each other such that each can accomplish its assigned statutory responsibilities while assuring at the same time adverse effects on Wisconsin's land, water, fish, and wildlife resources are minimized to the fullest extent practicable under the legislative mandates.

II. General Liaison, DOT Project Development Activities

Liaison between the departments on projects under consideration for development by DOT will be guided by the following:

A. DOT will provide DNR with copies of notices of intention to make changes in the State Trunk Highway System, notices of hearings scheduled for proposed changes, copies of annual proposed highway improvement programs, and copies of Federal and State Environmental Impact Statements. Other notices and documentation will be provided upon request.

B. DOT will inform DNR of the proposed new construction by providing copies of pertinent inter-departmental memoranda and preliminary plans indicating location and nature of work, immediately following authorization to incur engineering expenditures, to insure that DNR has this data at the earliest possible date.

C. DNR will review proposed improvements and make the recommendations necessary to comply with applicable environmental and regulatory requirements. DNR, in making its review and recommendations, will recognize that it is the policy of the state to provide a safe and economic transportation system with a minimal environmental impact.

D. DOT will give consideration to such DNR recommendations incident to the location, design, construction and maintenance of facilities. If DOT feels that it is not practicable to comply with the DNR recommendations, appropriate department staffs will meet and resolve any differences. In such considerations, both departments

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D. DOT will give consideration to such DNR recommendations incident to the location, design, construction and maintenance of facilities. If DOT feels that it is not practicable to comply with the DNR recommendations, appropriate department staffs will meet and resolve any differences. In such considerations, both departments

2. Whenever a District Office Liaison of either agency concludes there is a conflict which cannot be resolved by the respective agency staff involved in project review up to that point, he/she will notify the other agency's District Office Liaison in writing, describing the areas of conflict. The District Office Liaisons will meet in a timely manner and make every reasonable effort to resolve the conflict. If they are unable to reach agreement, they will prepare a written summary of the issue and remaining points of conflict, which will be hand delivered to their respective District Directors.

3. The District Directors will meet in a timely manner and make every reasonable effort to resolve the conflict. If they are unable to reach agreement, they will prepare a written description of the issue and the remaining points of conflict, which will be hand delivered or faxed to the appropriate Division Administrator(s) in their respective agencies. Steps 2 and 3 will be completed within a total of 21 days.

4. The Division Administrators, and no more than two additional representatives from each agency, will meet in a timely manner and make every reasonable effort to resolve the conflict. If the Administrators are unable to reach agreement within 14 calendar days, they will each notify their respective Secretary in writing.

5. The Secretaries will meet to resolve the conflict and may appoint, at any time, a mutually acceptable mediator to assist in resolving the conflict.

If the Secretaries decide to use a mediator, the mediator will have no authority to impose a settlement on either agency. The cost, if any, of a mediator will be borne by the agency initiating the conflict resolution process.

This step, whether a mediator is involved or not, is expected to be completed within 21 calendar days, unless extended by mutual agreement of the Secretaries.

The Secretaries will either agree on a specific solution to the conflict or will agree that the issues will be resolved through alternative means (or processes).

IV. <u>DNR Projects</u>

On those projects contemplated by DNR which will result in special land-use restrictions such as presently found in the Federal Land and Water Conservation Act (LAWCON) and the 1966 Federal DOT Act (Section 4(f)), DNR will inform DOT of such restrictions, if known, prior to committing action so that measures to provide for needed transportation corridors can be taken as much as is practical.

V. <u>Mutual Concurrence on Actions</u>

A. It is the intent of this agreement that joint review of projects will result in concurrence on the proper course of action to comply with the statutory obligations of each agency.

B. Actions by contractors–DOT usually implements its actions by letting contracts to the lowest qualified bidder. In these contracts the final product is usually specified in great detail, but the method of operations is left to the contractor's discretion. The climate of competitive bidding and relatively free choice of methods stimulates creativity and results in lower costs to the public. The contractors methods, however, are not specifically a part of the liaison and coordination described under II because the contractor is not known until the very last stages of action.

To insure that environmental regulations are compiled with in all applicable areas, such as stream crossings and wetland encroachments, DOT will require contractors to submit a plan of operation for review and approval by DOT. Further liaison with DNR will be necessary if the construction methods proposed in the operation plan have not been reviewed and concurred in by DNR during previous liaison on the project. Evidence of approval will be kept in the DOT engineers field office with a copy sent to the appropriate DNR district office. A contractor's operation which has been approved under this procedure shall be treated by DNR as an action by DOT.

C. Projects administered by the Division of Highways for other governmental units–DOT frequently administers transportation projects for counties, municipalities, and other local governing units as part of its statutory responsibilities. Those projects on which DOT exercises administrative control of plan preparation and contract supervision will be considered by DNR to be actions by DOT itself.

VI. <u>Construction Erosion Control</u>

Consistent with concepts and procedures outlined above and to meet each agencies responsibility under Act 416, Laws of 1983, it is the intent of each agency to cooperate to the fullest to minimize or eliminate construction erosion from DOT construction projects. Both agencies will implement the policy and procedures outlined in attachment A in meeting this objective.

VII. <u>Waterway Crossings and other Floodplain Encroachments</u>

A. <u>General Policy</u>:

Consistent with the above concepts, DOT recognizes that DNR has developed criteria specified in Administrative Codes NR 116 and NR 320 concerning floodplain encroachments, stream profiles, and navigational clearances. DOT concurs in-the spirit and intent of these Codes and will provide DNR and affected local units of government with information indicating the criteria used in the design and placement of structures in relation to the regional flood. DOT will cooperate fully with local units of government in their efforts to minimize flooding effects and to meet their responsibilities in floodplain zoning.

DOT considers discharge capacities, backwater elevations, potential upstream and downstream water damages, and protection of the roadway in the design of any water-related structure. DOT also considers land use and the property rights of present and future riparian and other affected property owners, upstream and downstream, consistent with the principle of just compensation.

B. Source, Distribution, and Use of Regional Flood Data:

For stream crossings involving new or replacement structures and for other floodplain encroachments, DOT shall compute the 100-year regional flood discharge and elevations in accordance with NR 116 and NR 320. In determining structure size and placement, DOT shall consider floodplain management standards pursuant to NR 116 and shall consider the hydraulic characteristics of the stream reach relative to existing impedances to flow. Upon completion of the design, predicted water surface elevations will be made available to the DNR, local unit of government, and the appropriate floodplain zoning authority. DOT or its authorized agent shall also notify affected property owners of the increase in the height of the regional flood as specified in NR 116 for the purpose of making appropriate legal arrangements with these property owners.

C. Appropriate Legal Arrangements:

DOT or its authorized agent shall ensure that appropriate legal arrangements have been made with affected property owners consistent with the Constitutional principle of just compensation. Such arrangements shall be commensurate with land use and with the amount of the increase in the height of the regional flood.

1. For streams that do not have floodplain zoning ordinances in effect:

   Appropriate legal arrangements shall consist only of written notification to affected property owners which informs them of the predicted increase in the height of the regional flood and of their rights under Section 88.87, Wisconsin Statutes. Evidence of this notification shall be provided to DNR by DOT or its authorized agent.

2. For streams that do have floodplain zoning ordinances in effect:

   Appropriate legal arrangements shall consist of written notification to affected property owners advising them of the change in the regional flood elevation on their property and, where consistent with the Constitutional principle of just compensation, of the acquisition of property rights or compensation prior to project construction for future damages through the purchase of flowage easements or other means of conveyance, through-condemnation as provided in Chapter 32, Wisconsin Statutes, or though any other legally enforceable document signed by an affected property owner which acknowledges the extent of increased flood elevations and the property owners' legal right to compensation. Written notification to affected property owners shall inform them further of their rights under Section 88.87, Wisconsin Statutes, to file a claim after damage occurs from unreasonable or unnecessary water accumulation resulting from bridge construction and its impedance of water flow and shall solicit their comments concerning the change in flood elevation.

Evidence of written notification; or where appropriate, evidence of the intent to acquire property rights, shall be provided to DNR by DOT or its authorized agent.

D. Relationship to Project Development:

DOT project development.scheduling normally provides sufficient lead time for the zoning ordinance amendment process to be completed prior to construction.

Upon notifying DNR, the local unit of government, and the appropriate floodPlain zoning authority of the predicted increase in the height of the regional flood, and making appropriate legal arrangements with affected property owners, and providing evidence of this to DNR. DOT or its authorized agent may proceed with project development.

DNR shall provide timely assistance to local units of government in the development, adoption, and administration of their official floodway lines, water surface profiles, floodplain zoning maps, and zoning ordinances consistent with their authority and responsibility under NR 116.

DNR shall notify DOT in a timely manner about any significant problems which might arise during the ordinance amendment process that might indicate reconsideration of the project development schedule. If such a situation arises, DOT and DNR shall resolve these jointly on an individual basis pursuant to the spirit and intent of this Agreement.

If a community fails to amend its ordinance in a timely manner (six months after the new regional flood elevation is made available to local officials and affected landowners) or denies the amendment, even though the new floodplain information has been provided and appropriate legal arrangements have been made with affected property owners, DOT may proceed with project construction after consultation with DNR.

VIII. Maintenance and Removal of Existing Structures

It is mutually recognized that DOT has the authority and responsibility to preserve the integrity of public-funded highways by means of a sound maintenance program. Also, the creation of a new highway often includes the removal of existing structures or roadbeds which are unusable or obsolete.

On normal planned highway maintenance and structure removal, DOT will maintain liaison with the DNR district office in the same manner as is set forth earlier in this agreement for construction projects. It is recognized that emergency maintenance activities necessitate expedited liaison procedures. In emergency maintenance situations, DOT will contact the DNR district office and furnish details on the project. However, the degree of notice furnished to DNR in emergency situations will be in direct correlation to the severity of the emergency. All efforts will be made by DOT to give as lengthy a notice as is possible. In emergency maintenance situations DNR will submit its recommendations on the project to DOT on an expedited basis.

DOT will maintain close liaison with DNR, as discussed throughout this agreement, to insure that the use of explosive does not result in damage to waterways, wetlands, and other environmentally sensitive areas nor result in the destruction of fish or game.

Wisconsin Department of Natural Resources

George E. Meyer

George E. Meyer, Secretary

8/15/95

Date

Wisconsin Department of Transportation

Charles H. Thompson

Charles H. Thompson, Secretary

8/15/95

Date

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Attachment A1
to the DOT/DNR Cooperative Agreement
Memorandum of Understanding
On Erosion Control and Storm Water Management

I. General.

DOT and DNR Administrations strongly endorse the following erosion control and storm water management practices and direct that they be included in design plans and carried out through field compliance. This Memorandum of Understanding (MOU) details the erosion control and storm water management practices approved by DOT and DNR.

II. Procedures.

1. ESTABLISH AND IMPLEMENT A CLEAR POLICY ON GOOD EROSION CONTROL AND STORM WATER MANAGEMENT PRACTICES:

DOT erosion control and storm water management policy and directives shall (1) stress the importance of and provide for effective and timely erosion control practices, and (2) inform staff of their responsibility for proper implementation and enforcement of erosion control policy and practices. Erosion control requirements and practices will be given added emphasis and enforcement during the construction season. The importance of erosion control and storm water management will be explained at appropriate meetings and conferences, including those with contractors. DOT will provide a presentation of updated erosion control and storm water management policies and practices as an item of discussion at appropriate annual district conferences. DOT will also include erosion control and storm water management on the agenda for the annual Contractor/Engineers Conference and for similar conferences with consultants and county highway managers.

2. DESIGNATE DISTRICT EROSION CONTROL SPECIALIST(S):

Assign the additional duties of advising and training other district staff about erosion control and storm water management matters to the district staff person(s) designated by the DOT District Director.

3. PROVIDE TRAINING FOR DOT STAFF TO INCREASE AWARENESS AND KNOWLEDGE OF EROSION CONTROL AND STORM WATER MANAGEMENT MEASURES:

Training will continue to be provided to DOT staff to increase staff awareness and knowledge of erosion control and storm water management applications and environmental concerns. DOT will consult with DNR when developing erosion

FHWA Hydraulic Engineering Publications
(www.fhwa.dot.gov/bridge)
February 10, 1999

The PUBLICATIONS are available from NTIS, National Technical Information Service, 5285 Port Royal Rd, Springfield, VA 22161, (703) 605-6000 (www.fedworld.gov/ntis). NO COPIES ARE AVAILABLE FROM FHWA

	HYDRAULIC DESIGN SERIES (HDS)	YEAR	FHWA-#	NTIS-#
HDS-1	Hydraulics of Bridge Waterways	1978	EPD-86-101	PB86-181708
HDS-2	Highway Hydrology (SI)	1996	SA-96-067	PB97-134290
HDS-3	Design Charts for Open-Channel Flow	1961	EPD-86-102	PB86-179249
HDS-4	Introduction to Highway Hydraulics (SI)	1997	HI-97-028	PB97-186761
HDS-5	Hydraulic Design of Highway Culverts*	1985	IP-85-15	PB86-196961

	HYDRAULIC ENGINEERING CIRCULARS (HEC)	YEAR	FHWA-#	NTIS-#
HEC-9	Debris-Control Structures	1971	EPD-86-106	PB86-179801
HEC-11	Design of Riprap Revetment	1989	IP-89-016	PB89-218424
HEC-14	Hyd. Design of Energy Dissipators for Culverts & Channels *	1983	EPD-86-110	PB86-180205
HEC-15	Design of Roadside Channels with Flexible Linings *	1988	IP-87-7	PB89-122584
HEC-17	Design of Encroachments on Flood Plains using Risk Analysis	1981	EPD-86-112	PB86-182110
HEC-18	Evaluating Scour at Bridges, Edition 3 (SI)	1995	HI-96-031	PB96-163498
HEC-20	Stream Stability at Highway Structures, Edition 2 (SI)	1995	HI-96-032	PB96-163480
HEC-21	Bridge Deck Drainage Systems	1993	SA-92-010	PB94-109584
HEC-22	Urban Drainage Design Manual (SI)	1996	SA-96-078	PB97-134308
HEC-23	Bridge Scour and Stream Instability Countermeasures (SI)	1997	HI-97-030	PB97-199491

	IMPLEMENTATION REPORTS (IMP)	YEAR	FHWA-#	NTIS-#
HIRE	Highways in the River Environment	1990	HI-90-016	PB90-252479
IMP	Underground Disposal of Storm Water Runoff, Design Guidelines	1980	TS-80-218	PB83-180257
IMP	Guide for Selecting Manning's Roughness Coef. for Natural Channels and Flood Plains	1984	TS-84-204	PB84-242585
IMP	Culvert Inspection Manual	1986	IP-86-2	PB87-151809
IMP	Structural Design Manual *	1983	IP-83-6	PB84-153485

	PUBLICATIONS ON CD-ROM **	YEAR	FHWA-#	NTIS-#
HDS-5	Hydraulic Design of Highway Culverts (CDROM), v1.00	1996	SA-96-080	N/A
	Installation and User's Guide (SI computation aids)	1996	SA-96-081	N/A

* Also available from McTRANS - 512 Weil Hall, Univ. of Florida, Gainesville, FL 32611-6585 (352) 392-0378, FAX (352) 392-3224, Messages 1-800-226-1013

** Available from Pallas, Inc - P.O. Box 3436, Logan, UT 84323-3446, (801) 755-0002 or "www.pallasinc.com"

FHWA Hydraulics Software List
(www.fhwa.dot.gov/bridge)
February 10, 1999

The software and related publications listed below are available from:

McTRANS - 512 Weil Hall, Univ. of Florida, Gainesville, FL 32611-6585, (352) 392-0378, FAX (352) 392-3224, Messages 1-800-226-1013 (www-mctrans.ce.ufl.edu)

PC-TRANS - 2011 Learned Hall, Lawrence, KS 66045, (913) 864-5655, FAX (913) 864-3199 (kuhub.cc.ukans.edu/~pctrans/index.html)

	TITLE	YEAR	MCTRANS	FHWA-#	NTIS-#
HY-7	Bridge Waterways Analysis Model	1986	WSPRO		
	WSPRO Research Report	1986	WSPRO.D	RD-86-108	PB87-216107
	WSPRO User's Manual (Version P60188)	1990	WSPRO.D	IP-89-27	PB91-218420
HY-8	FHWA Culvert Analysis (Version 6.0)	1996	HY8		
	Hydraulic Des. of Highway Culverts	1985	HY8.D	IP-85-15	PB86-196961
	Research Report (Version 1.0)	1987	HY8.D		
	HY 8 Applications Guide	1987	HY8.D	ED-87-101	NA
HY-9	Scour at Bridges (Version 5.0)	1994	SCOUR		
	HEC 18, Evaluating Scour at Bridges	1991	SCOUR.D	IP-90-017	PB91-198739
	HEC 20, Stream Stability at Highway Bridges	1991	SCOUR.DS	IP-90-014	PB91-198788
HY-10	BOXCAR (Version 1.0)	1989	BOXCAR		
	BOXCAR Users Manual	1989	BOXCAR.D	IP-89-018	PB90-115486
	Structural Design Manual	1983	BOXCAR.DS	IP-83-6	PB84-153485
	PIPECAR (Version 2.1)	1993	PIPECAR		
	PIPECAR Users Manual (Version 1.0)	1989	PIPECAR.D	IP-89-019	PB90-115478
	Structural Design Manual	1983	PIPECAR.DS	IP-83-6	PB84-153485
	CMPCHECK (Version 1.0)	1989	CMPCHECK		
HY-11	Preliminary Analysis System for WSP	1989	PAS		
	PAS USERS MANUAL	1989	PAS.D	IP-89-013	PB90-112723
HY-12	FESWMS-2DH (Version 1.0)	1989	FESWMS		
	FESWMS-2DH, Users Manual	1989	FESWMS.D	RD-88-177	NA
	FESWMS-2DH, Research Report	1989	FESWMS.DS	RD-88-146	PB91-106492
HY-TB	Hydraulic Toolbox (HEC 12, 14, & 15)	1989	HYDTOOL		
	HEC's 12, 14, and 15	1989	HYDTOOL.D		
CANDE	CANDE-89 (Version 1.0)	1989	CANDE		
	CANDE, Users Manual	1989	CANDE.D	RD-89-169	NA
HYDRAIN	Drainage Design System (Version 6.01)	1996	HYD6		
	HYDRAIN Users Manual (hard copy)	1996	HYD6.D	SA-96-064	NA
BRI-	Bridge Stream Tube for Alluvial River Sim.	1994	BRISTARS		
	BRI-STARS Users Manual (Version 3.3)	1994	BRISTARS.D		

Item 113 - Scour Critical Bridges 1 digit

Use a single-digit code as indicated below to identify the current status of the bridge regarding its vulnerability to scour. Scour analyses shall be made by hydraulic/geotechnical/structural engineers. Details on conducting a scour analysis are included in the FHWA Technical Advisory 5140.23 titled, "Evaluating Scour at Bridges." Whenever a rating factor of 4 or below is determined for this item, the rating factor for Item 60 - Substructure may need to be revised to reflect the severity of actual scour and resultant damage to the bridge. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to (1) observed scour at the bridge site or (2) a scour potential as determined from a scour evaluation study.

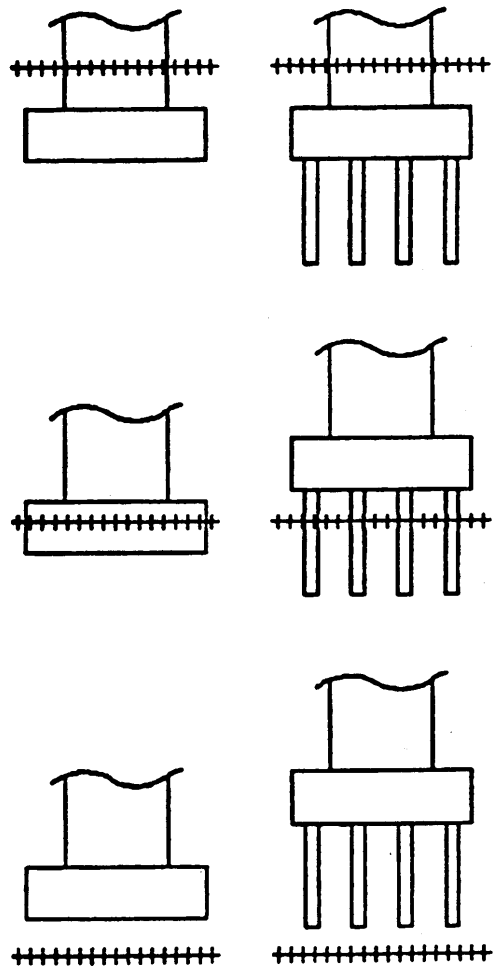
Code	Description
N	Bridge not over waterway.
U	Bridge with "unknown" foundation that has not been evaluated for scour. Since risk cannot be determined, flag for monitoring during flood events and, if appropriate, closure.
T	Bridge over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections. ("Unknown" foundations in "tidal" waters should be coded U.)
9	Bridge foundations (including piles) on dry land well above flood water elevations.
8	Bridge foundations determined to be stable for assessed or calculated scour conditions; calculated scour is above top of footing. (Example A)
7	Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical.
6	Scour calculation/evaluation has not been made. (<u>Use only to describe case where bridge has not yet been evaluated for scour potential</u>.)
5	Bridge foundations determined to be stable for calculated scour conditions; scour within limits of footing or piles. (Example B)
4	Bridge foundations determined to be stable for calculated scour conditions; field review indicates action is required to protect exposed foundations from effects of additional erosion and corrosion.
3	Bridge is scour critical; bridge foundations determined to be unstable for calculated scour conditions: - Scour within limits of footing or piles. (Example B) - Scour below spread-footing base or pile tips. (Example C)

(codes continued on the next page)

Item 113 - Scour Critical Bridges (cont'd)

Code	Description
2	Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations. Immediate action is required to provide scour countermeasures.
1	Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic.
0	Bridge is scour critical. Bridge has failed and is closed to traffic.

EXAMPLES:	CALCULATED SCOUR DEPTH	ACTION NEEDED
A. Above top of footing		None - indicate rating of 8 for this item
B. Within limits of footing or piles		Conduct foundation structural analysis
C. Below pile tips or spread-footing base		Provide for monitoring and scour countermeasures as necessary



SPREAD FOOTING (NOT FOUNDED IN ROCK)

PILE FOOTING

++++++++++++++++ = Calculated scour depth